"THE EFFECTS OF FREEZING AND THAWING ON THE ENGINEERING PROPERTIES OF CLAYS & SHALES".

by

W. E. Curtis - W. J. Dickson

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SYNOPSIS

Considerable research has been carried out on the mechanics of frost action in soils from which have evolved means of reducing or preventing frost heave. However, the effect of freezing and thawing on the engineering properties of soils is a matter which has received little attention to date. Studies which deal with the change in engineering properties which a soil may exhibit as a result of freezing and thawing have only recently been undertaken in the U.S.A.

In Canada at the present time the trend is toward all weather construction in order to minimize seasonal unemployment. Winter construction operations frequently demand that soil below new footings, which would normally be well below the depth of frost penetration, be exposed to freezing temperatures. It is important to know whether freezing and thawing will have any effect on the engineering properties of the foundation soil and subsequently on the stability of the new structure.

Possible effects of soil freezing and thawing are also of interest in Highway Engineering. The matter is being studied in the U.S.A., through field observations under the direction of the Highway Research Board, and through laboratory investigations under the direction of the Chief of Engineers of the U.S. Army.

This report covers tests carried out at the University of Alberta during the session 1954-55 in an attempt to determine the

effects of freezing and thawing on the engineering properties of clays and shales.

It would appear from results of these tests that freezing reduces the strength of clay soil, that some of the original strength is recovered with time and that the consolidation characteristics of the soil may be altered in a similar manner to remolding the specimens.

Although evidence is not conclusive, it further appears that freezing causes a shale to disintegrate and act like a cohesionless material under normal pressures when no change in moisture content is permitted.

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THE UNIVERSITY OF ALBERTA

THE EFFECTS OF FREEZING AND THAWING ON THE ENGINEERING PROPERTIES OF CLAYS AND SHALES

A DISSERTATION
SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES
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OF MASTER OF SCIENCE

FACULTY OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING

by
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and
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EDMONTON, ALBERTA

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DEFINITIONS AND SYMBOLS

Moisture Content (w) - is the ratio, expressed as a percentage, of the weight of water contained in a soil mass to the weight of the dry solid particles. A soil sample is considered to be dry when subjected to a temperature of 100°C to 110°C for six hours.

Deviator Stress - is the difference between the major and minor principal stresses in a soil at failure in triaxial compression.

Unconfined Compressive Strength - (q_u) is the maximum axial stress that a soil specimen will sustain in unconfined compression.

Consolidation Curve - This term as used herein refers to the voids ratio versus logarithm of pressure curve obtained in one-dimensional consolidation.

Virgin Branch of Consolidation Curve - refers to the straight line portion of the above mentioned consolidation curve obtained at pressures well in excess of the pre-consolidation pressure for the soil in question.

Frost Heave - is the raising of the surface of the soil as a result of the accumulation of ice lenses in the underlying soil.

Frost - Susceptible Soils - are those in which significant ice segregation will occur when moisture is available and the requisite freezing conditions are present.

Frost Action - is a term used in referring to freezing of moisture in materials and the resultant effects of these materials and on the structures of which they are a part.

Ice Segregation - in soils is the growth of bodies of ice during the freezing process, most commonly as ice lenses or layers oriented normal to the direction of heat loss, but also as veins and masses having other patterns.

Degree of Saturation - is the ratio expressed as a percentage of the volume of water in a given soil mass to the total volume of voids.

Ground-water Table - is the free-water surface nearest to the ground surface.

Capillarity - is that property which enables a soil to draw and hold water above the elevation at which atmospheric pressure exists in the water.

Overburden Pressure - is the force exerted at any given point in a soil by the weight of the overlying material.

Closed System - is the condition where no free water is made available from outside the specimen during the freezing process.

Open System - is a condition where free water is made available from outside the specimen during the freezing process.

Effective Overburden Pressure - is the force exerted at a given point in a soil by the submerged weight of material between the point in question and the water table, plus the full weight of soil and soil moisture located above the water table.

Aggregation - is the formation of secondary soil particles from primary soil particles, (i.e. sand, silt or clay) held together by some cementing agent.

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CHAPTER I

REVIEW OF LITERATURE

It has long been recognized by Engineers in Canada, the northern states of U.S.A., and Europe that the serviceability of roads and airfields varies with the seasons. During the winter months, roads built on frost susceptible bases and subgrades are subject to frost heaving with ensuing hazards to fast moving traffic. The load bearing capacity of such roads is high while the ground remains frozen. However, during the spring break-up, the same roads become soft with development of ruts, isolated frost boils or pumping under rigid pavements.

Considerable research has been carried out in Europe, primarily in Sweden, and in the U.S.A., in an attempt to explain the phenomenon of frost action in soils. The most prominent workers in this field have been Fil dr Gunnar Beskow, working for the Swedish Geological Society, and Stephen M. Taber, a geologist in the U.S.A.

A summary of Beskow's work was published under the auspices of the Swedish Geological Society in 1935 entitled Soil Freezing and Frost Heaving with special Application to Roads and Railroads. This publication was translated into English by J.O. Osterberg at Northwestern University along with a supplement of progress from 1935 to 1946.

The Highway Research Board of the N.R.C., U.S.A., prepared and published a review of literature (1765-1951) in 1952 (Ref. 1) and the proceedings of a symposium dealing with frost action held under its own auspices in 1952 (Ref. 2).

The three publications mentioned above provide an exhaustive review of the research work carried out until 1950 and it would be impossible to relate more than a brief mention of the highlights of this work in the space available here. The brief review given below is drawn chiefly from Ref. 1.

As far back as 1765 Runeberg in Sweden found clear ice in frozen soil but thought heaving was due only to the expansion of the contained water. In 1897 Holmquist observed that in clay soils, the soil between the lowest ice layers was often unfrozen thus indicating a lowering of the freezing point when water was withdrawn from the soil. In 1905 Becker and Day found that growing crystals in a saturated solution of alum were capable of raising a kilogram weight. In 1916 Taber suspected that the same principle applied to the formation of ice columns and later proved that ice forming in clay soils was capable of lifting weights. He then concluded that frost heaving was caused by the growth of ice crystals into lenses or layers of ice.

During period 1914 to 1923, Bouyoucos through extensive studies found a difference in freezing point in different soils, and a difference in freezing point in the same soils which had different moisture contents.

Bouyoucos (1921) as well as Zunker (1933) and Ledideff (1927), all suggested that the forces responsible for water movements were analgous to those which move water in a capillary tube. Buckingham (1907) had previously held a different view. He suggested that water flowed from a saturated area to a dry area in a manner analgous to the flow of electricity from a high potential to a low potential.

As a result of the work of Holmquist and Bouyoucos, Taber conceived the idea that water continues to flow in some cases in a freezing soil and nourishes the growing ice crystals. In 1929 he proved conclusively what is now generally accepted as the theory of frost action in soils which follows.

All soil water does not freeze at the same temperature which makes it possible for water to move to the growing ice crystal. The growing ice crystal displaces the material overlying it and thus developes ice lenses or strata. The frost line is relatively stationary during the growth of the crystals, and the continuation of the growth of the crystals produces ground heaving.

Later work by A. Casagrande indicates that the grain size of the soil has a bearing on the severity of frost action. A soil, to be susceptible to frost heaving, must contain sufficient fines to induce capillary action and upward movement of water from below. At the same time the soil must not be so fine that its permeability is

too small to permit flow of water.

A second theory has been proposed by Benklman and Olmstead of the Michigan State Highway Department, which is generally incompatible with conditions, especially in Canada, and is therefore not widely accepted. Their hypothesis supposes that upon one cycle of freezing and thawing, the associated expansion and contraction of the pore water creates a vacuum in the soil voids following thaw. Water is then supposedly drawn into the voids and subsequent refreezing and rethawing continues to draw water into the soil and to build up layers of ice with ensuing frost heaving.

The generally accepted theory of frost action is attributed to Taber, and his findings along with those of other investigators, are given by Winn in the following summary (Ref. 1).

(1) Destructive freezing is associated with the formation of segregated ice.

(2) Total frost heave is equal to the sum of the total thicknesses of the ice lenses.

(3) Total frost heave is proportional to the increas in total water content of frozen soil.

(4) Soil must be in state of capillary saturation for ice segregation to take place.

(5) A supply of water must be available either in the soil or from some external source.

(6) For normal conditions, the percentage of 0.02 mm diameter grains is critical.

(7) One slow freeze will cause heaving. Thawing and re-freezing increases severity but does not change the basic action.

(8) The cumulative curve of degree-hours of freezing vs time is a qualitive measure of increase of heave with time.

(9) The following conditions are ALL necessary for ice *segregation and frost heave;

(i) Capillary saturation of the soil at the beginning and during freezing.
(ii) A free supply of water from within or without.
(iii) A minimum percentage of grains smaller than 0.02 mm (3 to 10).
(iv) Gradual decrease in temperature of the air above the soil to below freezing temperature.

The effects of freezing and thawing on soils was also being studied by physicists and agriculturists. Baver (1940, Ref. 3) found that alternate freezing and thawing was a factor in developing soil structure, causing a granulating action on clods that was more effective than drying and wetting. He cites the studies of Jung in which was found that freezing may cause either **aggregation or dispersion, the nature of the crystallization of the ice being the determining factor. With slow cooling the ice crystals serve as growth centres to which water is drawn from around the particles, making possible more intimate contact between soil grains. Rapid cooling

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* Segregation is the separation of the soil particles by the formation of ice crystals or ice lenses in the soil.

** Aggregation is the formation of secondary soil particles from the primary soil particles (sand, silt or clay) held together by some cementing agent.

causes breaking up of aggregates since large number of small crystals are formed. The water content is as important as the rate of cooling. Aggregation increases under slow cooling with increase in moisture content up to about 50^{o}/o saturation.

Gardner (1945, Ref. 4) found that ice crystallization had a dehydrating and densifying effect on semi-dispersed soils which developed a stable flaky structure in soils high in replaceable calcium and an unstable flaky structure in soils high in replaceable sodium. Freezing and thawing did not restore permeability in sodium saturated soils nor did replacement of sodium ions with calcium ions without freezing. However, after calcium chloride was added, freezing did effect an increase in permeability. Freezing and thawing aided in restoring permeability and structure in soils injured by sodium salts.

Having obtained a satisfactory explanation of the mechanics of frost heaving with subsequent break up in the spring, engineers then became interested in loss of bearing capacity generally in roads not necessarily susceptible to severe frost heaving. Work was begun in 1944 at the Frost Effects Laboratory, New England Division, Corps of Engineers, U.S. Army, for the Airfields Branch, Office of the Chief of Engineers. The objective of that program was chiefly to determine the effects of frost action on the load carrying capacity of airfields (Ref. 2 pp 246). As a result of field studies,

the need for comprehensive laboratory investigations became apparent and a coldroom was constructed in the Frost Effects Laboratory in January, 1950. A research program was planned to determine;

(1) Effect of particle size distribution on ice segregation in soils;

(2) Effect of degree of compaction on ice segregation in frost susceptible soils;

(3) Effect of initial degree of saturation on ice segregation in a frost susceptible soil in a closed system;

(4) Effect of surcharge or overburden pressure on ice segregation in frost susceptible soils;

(5) * Effect of alternate freezing and thawing on permanence of initial compacted density and strength of soils;

(6) Effect of soil properties such as voids ratio, permeability and capillarity on ice segregation;

(7) Effect of proximity of water table on ice segregation in frost susceptible soils;

(8) Effect of frost melting on strength characteristics of soils, including time required for weakened soils to return to normal strength after thawing;

(9) Effect of admixtures in preventing or retarding the formation of ice lenses in frost susceptible soils;

(10) Effect of the chemical nature of the soil minerals and of the dissolved salts in the pore water on ice segregation;

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* Underlining indicates points in the above program closely associated with work on this thesis.

(11) Frost susceptibility or non-frost susceptibility of base and subgrade soils from various air-fields in the northern U.S.A.;

(12) The nature of physical laws governing ice segregation in soils, an understanding of which is needed to aid the development of design criteria;

A progress report of these activities was published in 1953 by the Highway Research Board as Bulletin 71 entitled Soil Temperature and Ground Freezing (Ref. 7).

Concurrently, field studies of the effects of freezing and thawing on the load carrying capacity of roads was being carried out by several state highway departments in the U.S.A. The results of these studies to date have been published by the Highway Research Board as Bulletins 40, 54 and 96, entitled Load Carrying Capacity of Frost Affected Roads (Ref. 5, 6 and 8).

It has been found that the bearing capacity of roads fluctuates with the seasons and follows a fairly definite annual pattern. The percentage of loss of strength varies with the severity of the winter; but the time of year at which a road exhibits a given percentage of its peak capacity is almost independent of the location. In other words, a road generally is weakest in March or April reaching a peak strength during October, November, December and January.

Of particular interest in this thesis are items 5 and 8 of the above mentioned program being carried out at the Frost Affects Laboratory. Unfortunately detailed results of these tests have not been published to date.

The latest Highway Research Board publication on this subject, Bulletin 100, entitled Soil Freezing (Ref. 9), offers a discussion by Prof. E.J. Yoder, Purdue University, on results of tests performed on remolded soil specimens containing admixtures of calcium chloride in order to determine what effect that chemical may have in reducing the loss of strength of the soil as a result of freezing and thawing. Two freezing temperatures were used: 24°F and -18°F. At the same time control tests were made on samples containing no calcium chloride. Some samples were frozen while others were kept at room temperature during the cycle in which other specimens were frozen. Twenty-four hours of reduced temperature and twenty-four hours of thawing constituted one cycle of freezing and thawing.

The results of the control tests, i.e., on specimens containing no calcium chloride, indirectly offer a comparison with the results of tests carried out at the University of Alberta. A loss of strength of specimens as a result of freezing was obtained in both testing programs.

It should again be stressed, however, that the tests reported

upon by Prof. Yoder employed remolded specimens and that the soil was allowed to absorb water during thawing, while the tests carried out at the University of Alberta for this thesis employed undisturbed samples in a closed system.

CHAPTER 2

ORGANIZATION AND SCOPE OF INVESTIGATION

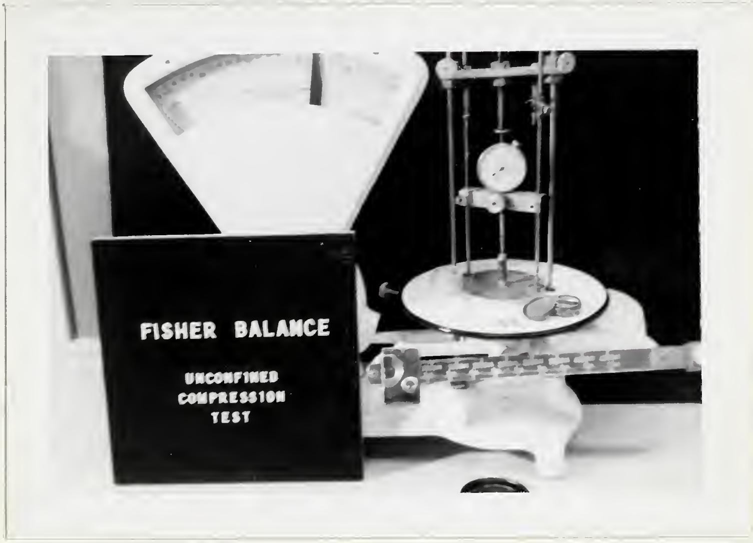
This project was concerned chiefly with determining the effects of freezing and thawing on strength characteristics of a soil, but was extended to include a minor investigation of the effects on the consolidation characteristics of the soil.

The materials were selected on the basis of homogeneity and uniformity of engineering properties within each type, and the laboratory investigations were divided into three groups as follows;

A. Classification tests to determine the engineering properties of the soils used in the various tests.

B. Strength tests to determine the validity of using 1 cm. diameter size, as opposed to 1.4 in. diameter size specimens for economy; and subsequently to determine whether there is a loss of strength of clays, silty clays and shales as a result of either rapid or slow freezing and thawing in a closed system. Should the foregoing tests indicate that there was a loss of strength, it was planned to determine any subsequent recovery of strength with time after the freezing and thawing process. The strength tests on 1 cm. specimens were carried out using the Fisher balance apparatus illustrated on Plate 1.

C. Consolidation tests were carried out on unfrozen, rapid frozen and remolded specimens to determine whether freezing and



Picture of the FISHER BALANCE APPARATUS with 1 cm. diameter specimen in position for testing.

thawing affected the compressive index and rate of consolidation in any way.

Freezing and thawing of specimens was carried out in a closed system in all cases, so that no moisture could enter or leave the soil during the process. Care was taken to seal the specimens in waxed metal containers to prevent or retard evaporation. This condition would correspond to the field situation where the soil is located above the water table a distance greater than the height of capillary rise. In this case there could only be local migration of the natural soil moisture during freezing so that the formation of ice lenses and frost heaving would not occur to any extent. The total expansion of the soil would be due only to the expansion of the soil moisture upon freezing.

CHAPTER 3

THEORETICAL CONSIDERATIONS *

Since 1929 when Taber provided a satisfactory explanation of the mechanics of Frost Action, further research work in the U.S.A., has revealed many of the factors which influence Frost Action.

According to Johnson and Lovell, (Ref 7) these factors can be divided into extrinsic and intrinsic considerations. The former have to do with loads imposed on the soil and the climate to which it is subjected. The latter has to do with the physical and chemical properties of the soil and the state of the soil mass. This discussion will be limited to the intrinsic considerations.

According to Johnson and Lovell the intrinsic factors include the following:

1. State of the soil mass
    - (a) Moisture content and distribution.
    - (b) Porosity and volume weight.
    - (c) Structure
    - (d) Temperature

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* Pages 14 to 19 include a brief summary of the knowledge to date regarding the effects of freezing on the engineering properties of soils. This summary is drawn freely from all references listed in the bibliography.

2. Composition of the soil
    - (a) Chemical Composition
        - (i) Mineral
        - (ii) Organic content
    - (b) Physical Composition
        - (i) Grain shape
        - (ii) Grain size

3. Physical Properties of the soil
    - (a) Resistance to load deformation
        - (i) Adfreezing
        - (ii) Strength of frozen soil
        - (iii) Reduced strength of thawed soil
    - (b) Moisture movement and retention
        - (i) Capillarity
        - (ii) Permeability
        - (iii) Vapor movement
        - (iv) Retention characteristics
    - (c) Volume change
        - (i) Swell (heave)
        - (ii) Shrinkage

4. Thermal Properties
    - (a) Specific heat
    - (b) Heat capacity
    - (c) Conductivity
    - (d) Diffusivity
    - (e) Latent heat of soil water

It appears that moisture content of soil at the beginning of freezing largely determines the amount of segregated ice and the heaving of the soil on freezing. The increase in the amount of soil moisture and change in its distribution through the soil, plus the changes in soil density and structure resulting from ice segregation, determine the magnitude of the reduction in load carrying capacity on thawing. Data to date suggest that no detrimental frost action results if the initial moisture content is less than 65 percent of saturation. Johnson and Lovell state "An outside source of free ground water is not a requisite for frost action in soils. However, the availability of free ground water near the zone of freezing greatly intensifies all phases of frost action".

For saturated soils or soils with a degree of saturation greater than 65%, Since frost action depends upon the amount of water available and the rate at which it can be drawn to the freezing zone, the severity of frost action varies inversely with the initial soil density.

The structure of a soil influences the permeability and subsequently the susceptibility of the soil to frost action. Frost action in turn may modify soil structure. Freezing and thawing may cause aggregation of the soil due to localized migration of moisture. It should be expected that aggregation would be most pronounced following slow freezing. As a consequence of aggregation, fissures may form in the soil mass which would admit free ground water. This phenomenon is commonly referred to as the formation of nugget structure

and occurs primarily in clay type soils.

The chemical composition of the soil particles determines the capacity of the soil to adsorb water and to prevent movement of free water. For example, Montmorillonite clays are less susceptible to ice segregation than Kaolinite clays; and both are much less susceptible than silty soils.

The grain size of the soil particles has a bearing on ice segregation in that fine grained soils have a higher voids ratio and higher capillarity. As a result, the fine grained soils are capable of holding more moisture and raising moisture higher above the water table than do coarse grained soils. There is a limitation to this capacity set by the chemical composition as previously discussed.

The rate at which the frost line penetrates into a soil depends upon the heat conductivity of the soil and upon the freezing temperature of the water.

Although the above mentioned factors are known to influence frost action in soils, quantitative determinations have still to be made. It should, however, be expected that a soil which has been frozen and thawed would exhibit a reduction in strength in unconfined compression tests. Dispersion or aggregation and fissuring of the soil can result from freezing and thawing, the extent of which in a closed system would depend largely on the freezing conditions and on the degree of saturation of the soil.

Under rapid freezing conditions, a saturated soil is likely to be dispersed, losing cohesion and subsequent unconfined compressive strength. The same soil may suffer a smaller loss of strength in triaxial compression because of the confining lateral pressure.

Under slow freezing conditions, a saturated soil is likely to experience aggregation and fissuring. The unconfined compressive strength is likely to be reduced even further than it would be following rapid freezing. Once again the reduction in strength in triaxial compression is likely to be less than in the unconfined state. However, the strength of the soil is likely to be determined by its internal frictional resistance along fissured planes, and would likely be less following slow freezing than it would be following rapid freezing.

Disturbance of a soil as a result of freezing and thawing might be expected to affect the consolidation characteristics of the soil as does remolding.

Apparently remolding a soil may have two separate effects on the soil's consolidation characteristics. In the first place, remolding is likely to destroy the bond stress (described by Terzaghi, Ref. 11) between particles created by pre-consolidation, and so produce an increase in voids ratio at zero pressure.

Secondly, remolding may cause a collapse of the soil structure and so produce a reduction in voids ratio at zero pressure. The result

of the two effects would be an increase in voids ratio, but it is unlikely that the initial voids ratio before consolidation would be attained, since the collapse of any structure the soil may have would reduce the voids ratio below the latter value.

According to Peck, Hanson and Thornburn, (Ref. 10), the voids ratio versus logarithm of pressure curve obtained in one-dimensional consolidation for a remolded soil is displaced below the virgin consolidation curve for the same soil. Further, the two curves intersect at zero voids ratio. Consequently, the compressive index for the remolded soil is smaller than that for the same soil undisturbed.

It is felt that freezing of a soil, resulting in expansion of the pore water, would also partially destroy the bond created by pre-compression; but that it may have less effect on the orientation of particles and hence on soil structure.

Based on this reasoning, it is suspected that freezing of a soil may have an effect similar to remolding on its consolidation characteristics, but that the associated reduction in compressive index may be smaller.

CHAPTER 4

SELECTION AND SAMPLING OF MATERIALS

The following samples from the Edmonton area were used in this testing program:

(1) Med-Dent clay obtained from the Medical-Dental Centre on Jasper Avenue west of the C.P.R. underpass.

(2) Failing clay from the Failing Drilling Company property at the corner of 109th Street and Kingsway.

(3) McKernan Silty clay from McKernan Lake School at the corner of 114th Street and 76th Avenue.

(4) Power House shale from the City Power Plant Annex on 105 Street.

Material included a medium plastic silty clay from sites 1 and 3, referred to as Med-Dent clay and McKernan silty clay respectively, as well as an highly plastic clay from site 2 to be known as Failing Clay, and a dense bentonitic shale from site 4 to be known as Power House shale.

Considerable care was exercised in all cases to obtain relatively undisturbed samples. The field sampling method is described below.

Test holes were drilled by means of a 4" diameter hand auger and samples were taken at various depths with 3" diameter thin walled Shelby sampling tubes. Following the removal of each

sample, the hole was cleared with the auger and the disturbed material was recovered for classification tests and soil moisture determination. A soil moisture profile was prepared for each hole as shown on Plates 2 to 5 inclusive.

The undisturbed samples were completely waxed in the 3 in. diameter Shelby sample tubes, labelled and stored in a moist room for future testing.

CHAPTER 5

CLASSIFICATION TESTS

Preliminary classification tests were carried out on each type of soil used in accordance with A.S.T.M. Standards for Testing Soils. These tests included specific gravity determination, Atterberg limits and grain size analysis. The results of the tests are plotted of soil profiles shown on Plates 2 to 5 inclusive and on the data sheets contained herein.

UNIVERSITY OF ALBERTA,

EDMONTON, ALBERTA.

SOIL PROFILE AND SUMMARY

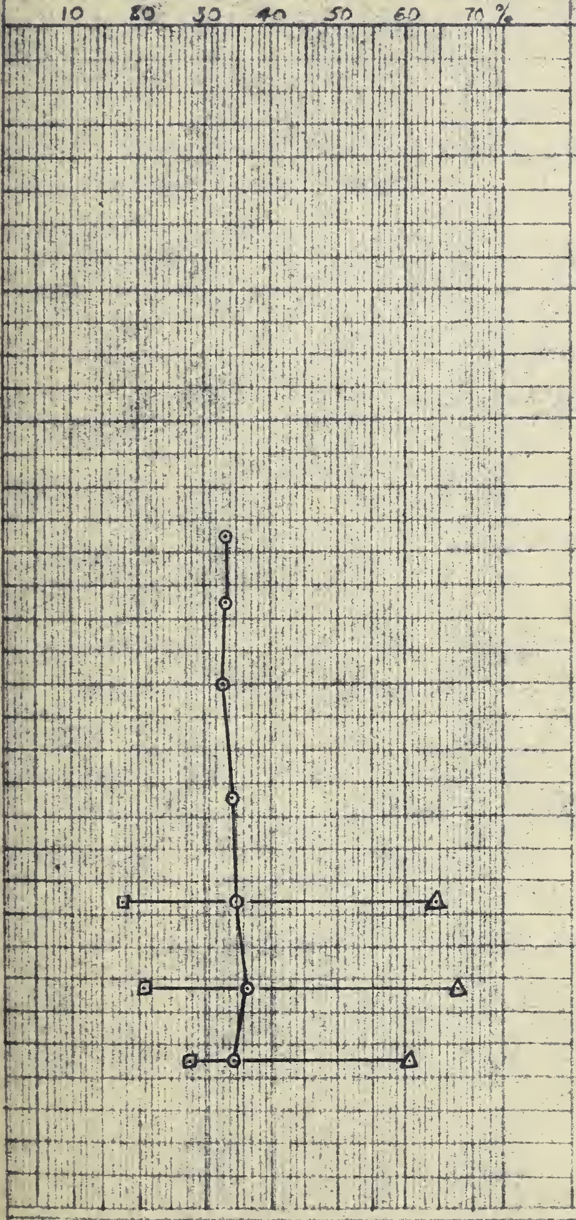
OF LABORATORY TESTING.

PROJECT - THESIS.

STUDY OF THE EFFECT OF FREEZING & THAWING ON THE ENGINEERING PROPERTIES OF CLAYS AND SHALES.

DATE MARCH 24 1955.

PLOT OF ATTERBERG LIMITS AND NATURAL MOISTURE CONTENTS.



DEPTH FT.	HOLE 1 McKERNAN LAKE SITE GROUND SURFACE	MOIST CONT. %	I_P	G_S
	Silty overburden			
2				
4	Brown, highly plastic, silty clay of medium to stiff consistency. Moisture content slightly above the plastic limit and increasing slightly with depth. Uniform material to the bottom of the hole.			
6				
8		33.5		
		33.5		
10		33.0		
		34.7		
12				
		35.0	46.7	
14				
		36.3	47.1	2.73
16		34.8	33.0	
	Bottom of hole at 16'9"			

-NOTE -

□ Plastic Limit.
△ Liquid limit.
⊙ Natural moisture content.

G.W.L. indicates ground water level at the time the test hole was drilled.

PLATE 2

UNIVERSITY OF ALBERTA,
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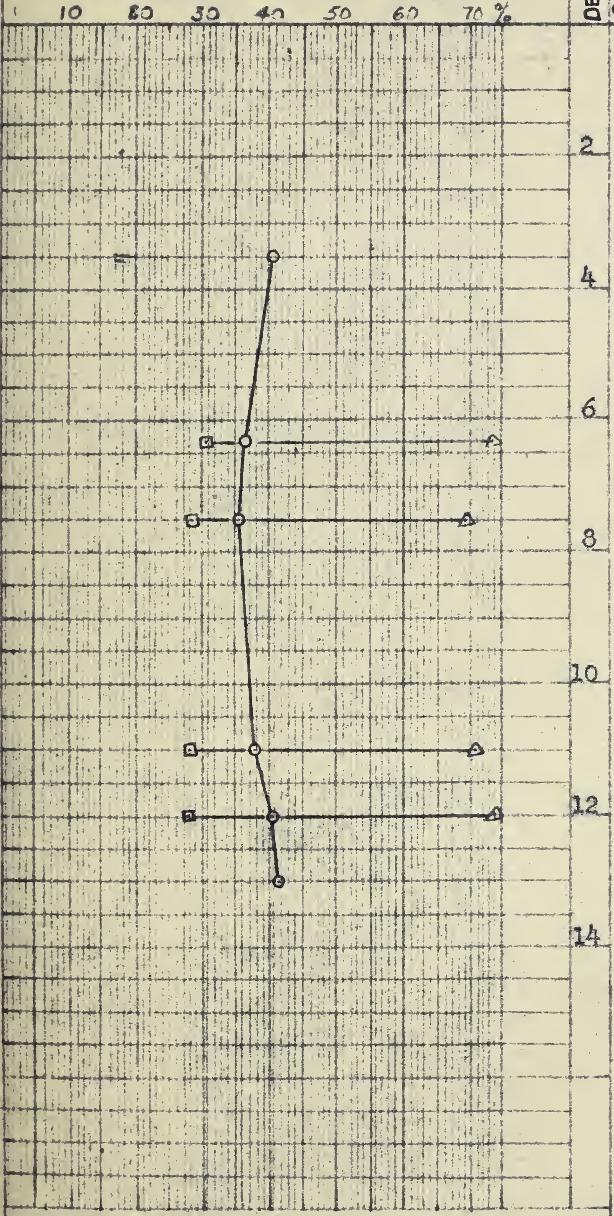
SOIL PROFILE AND SUMMARY
OF LABORATORY TESTING.

PROJECT — THESIS.

STUDY OF THE EFFECT OF FREEZING & THAWING ON THE ENGINEERING PROPERTIES OF CLAYS AND SHALES.

DATE March 24 1955.

PLOT OF ATTERBERG LIMITS
AND NATURAL MOISTURE CONTENTS.



HOLE 2
FAILING DRILLING SITE.
GROUND SURFACE →

SAMPLE	DESCRIPTION	MOIST CONT. %	I_P	G_S
	Silty - clay topsoil	40.5		
	Black, medium plastic clay changing to gray - brown in color. Medium consistency & with nugget structure evident.	36.5	43.1	
	Gray, highly plastic clay of stiff consistency. Moisture content increasing with depth. Material exhibits soft consistency on remoulding. (very thin water bearing sand strata at depth of 5')	35.6	40.3	
		37.8	42.7	2.71
		40.2	46.3	
		41.7		
	Bottom of hole at 13'			

-NOTE -
□ Plastic Limit.
△ Liquid Limit.
⊙ Natural moisture content.

G.W.L. indicates ground water level at the time the test hole was drilled.

PLATE 3

UNIVERSITY OF ALBERTA,

EDMONTON, ALBERTA.

SOIL PROFILE AND SUMMARY

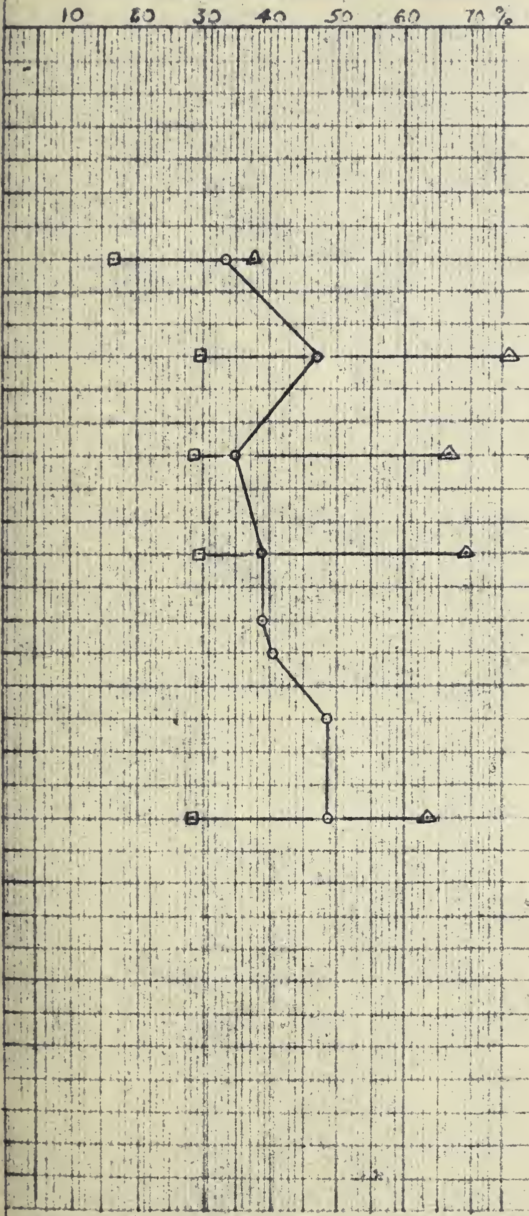
OF LABORATORY TESTING.

PROJECT - THESIS.

STUDY OF THE EFFECT OF FREEZING ON THE ENGINEERING PROPERTIES OF CLAYS AND SHALES.

DATE March 24 1955.

PLOT OF ATTERBERG LIMITS AND NATURAL MOISTURE CONTENTS.



DEPTH FT.	HOLE 3 FAILING DRILLING SITE. GROUND SURFACE →	MOIST CONT. %	I_P	G_S
	Silty clay topsoil.			
2				
	Medium plastic, black clay changing to brown in color. Medium consistency, nugget structure.	30.42	21.0	
4				
		46.9	46.7	
6				
	Highly plastic gray clay of stiff consistency. Moizture content above the plastic limit and increasing slightly with depth.	35.0	38.0	
8		38.3	40.4	2.74
		38.4		
		40.1		
10				
		48.8		
12		48.3	35.1	
	Bottom of hole at 12'6"			
14				

-NOTE -

□ Plastic Limit.
△ Liquid Limit.
⊙ Natural moisture content.

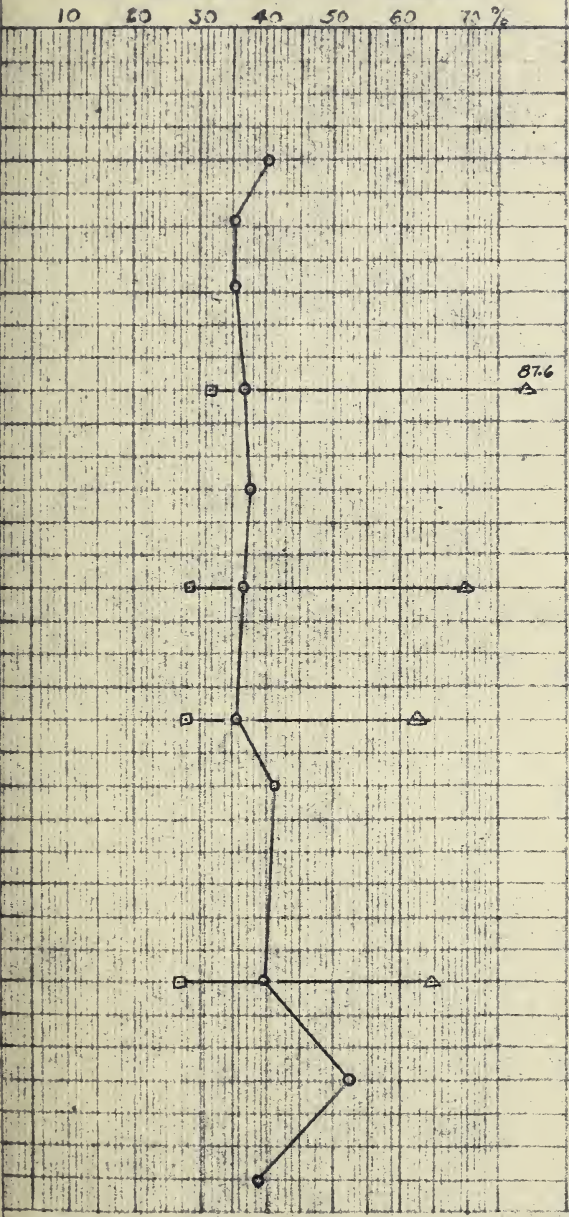
G.W.L. indicates ground water level at the time the test hole was drilled.

PLATE 4

UNIVERSITY OF ALBERTA,
EDMONTON, ALBERTA.
SOIL PROFILE AND SUMMARY
OF LABORATORY TESTING.

PROJECT - THESIS.
STUDY OF THE EFFECT OF FREEZING & THAWING ON THE ENGINEERING PROPERTIES OF CLAYS AND SHALES.
DATE March 24, 1955.

PLOT OF ATTERBERG LIMITS AND NATURAL MOISTURE CONTENTS.



DEPTH FT.	HOLE 4 FAILING DRILLING SITE GROUND SURFACE →	MOIST CONT. %	I_P	G_S
	Silty clay fill. Organic material.			
2	Soft, plastic, gray clay of medium consistency.	40.2 35.1		
4	Highly plastic gray clay, with small roots throughout.	35.1		
6	Thin layer of water bearing sand.	36.7	55.8	
8	Soft, highly plastic, gray clay. Slightly wetter than the plastic limit in undisturbed state. Material increasing slightly in moisture content with depth.	37.2		2.70
		36.3	41.5	
10		35.7	34.4	
12		41.1		
14		39.8	37.9	
16		52.7		
18	Bottom of hole at 17'6".	38.1		

-NOTE -
□ Plastic Limit.
△ Liquid Limit.
⊙ Natural moisture content.

G.W.L. indicates ground water level at the time the test hole was drilled.

PLATE 5

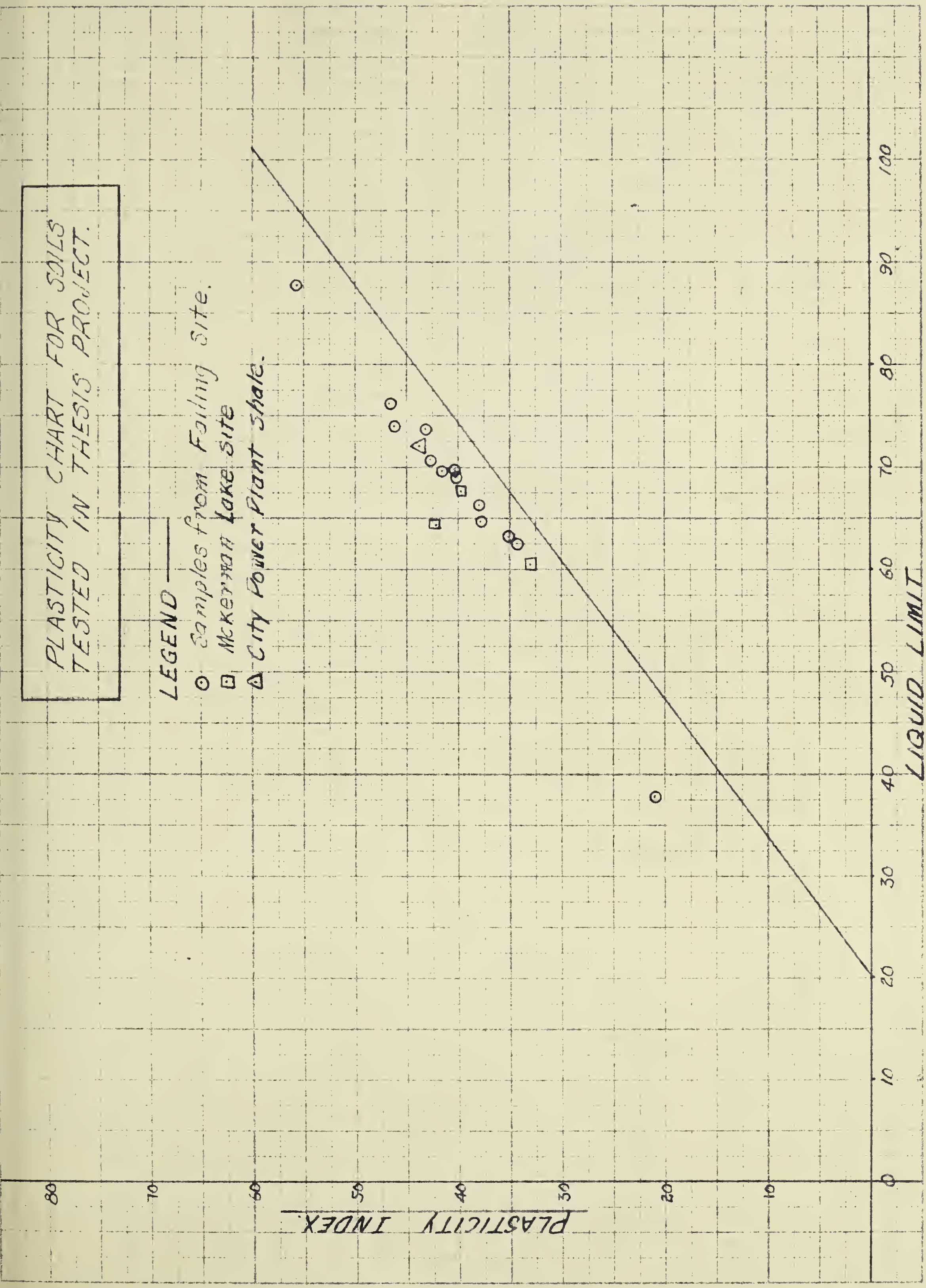

PLASTICITY CHART FOR SOILS TESTED IN THESIS PROJECT.
LEGEND —
⊙ Samples from Failing Site.
⊡ McKerran Lake site
△ City Power Plant Shale.
LIQUID LIMIT
0
10
20
30
40
50
60
70
80
90
100
PLASTICITY INDEX
0
10
20
30
40
50
60
70
80

PLATE 6

UNIVERSITY of ALBERTA
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SOIL MECHANICS LABORATORY
GRAIN SIZE CURVE

PROJECT	
SITE	
SAMPLE	
LOCATION	
HOLE	DEPTH
TECHNICIAN	DATE

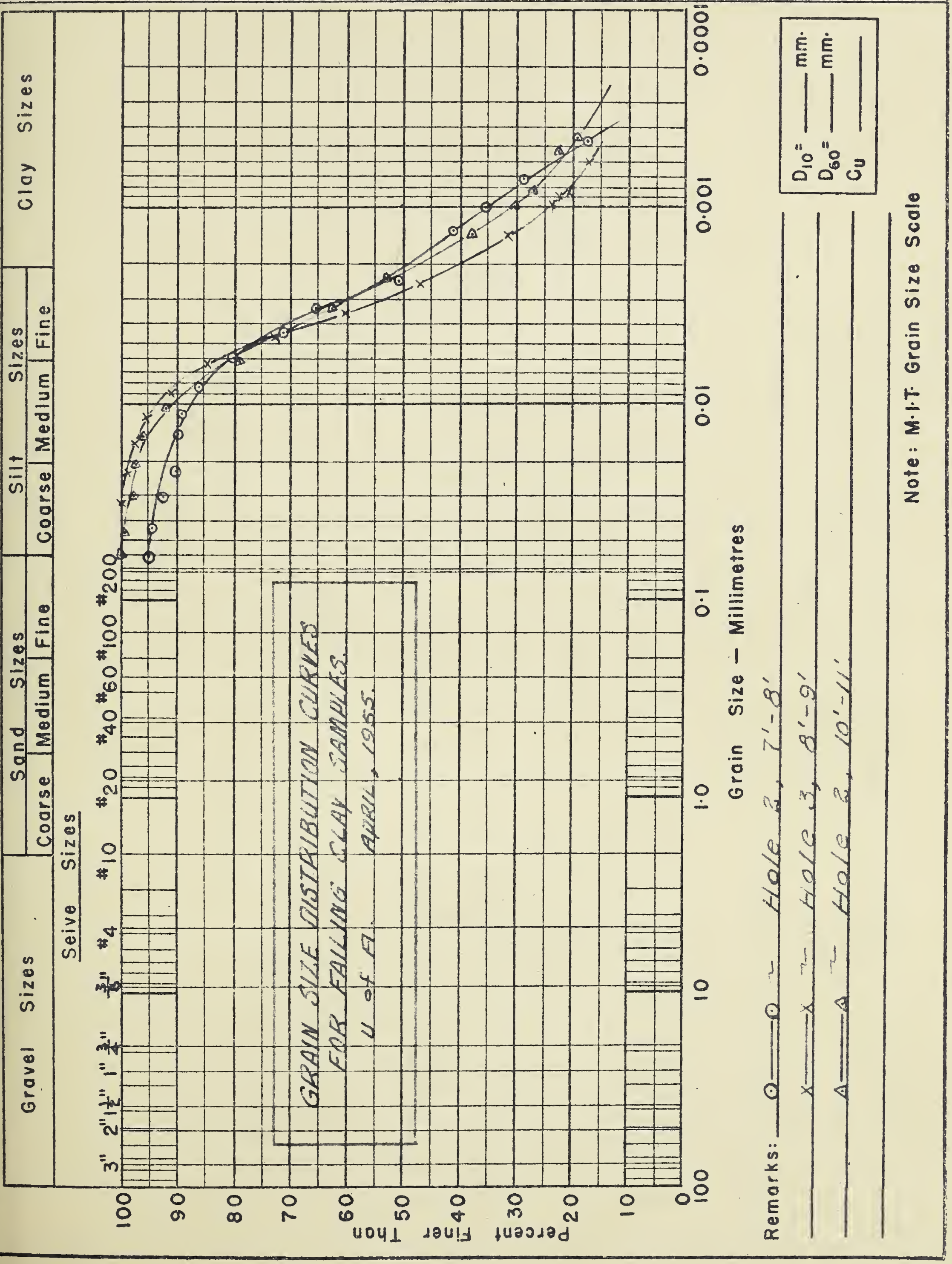
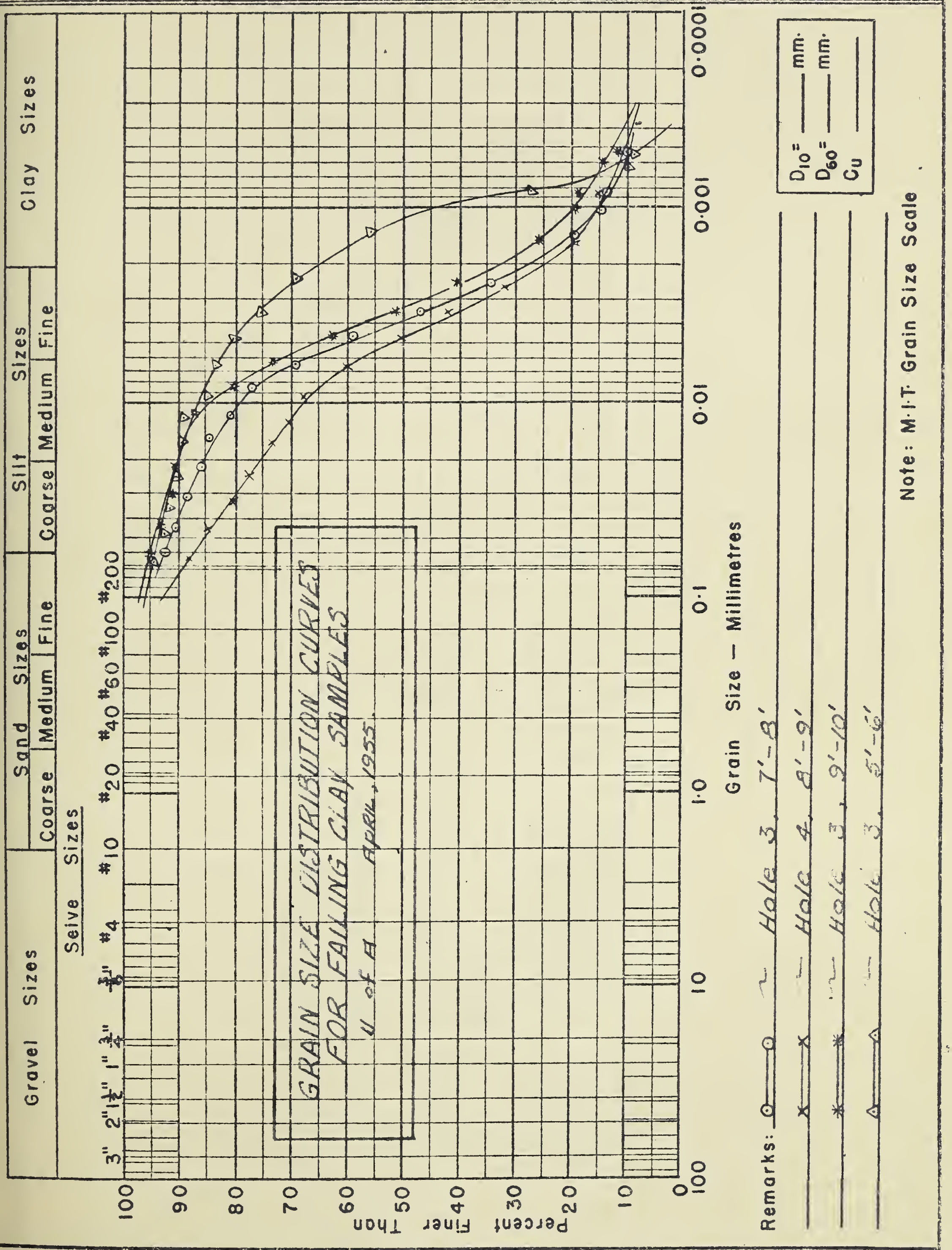


PLATE 7.

UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

GRAIN SIZE CURVE

PROJECT	
SITE	
SAMPLE	
LOCATION	
HOLE	DEPTH
TECHNICIAN	DATE



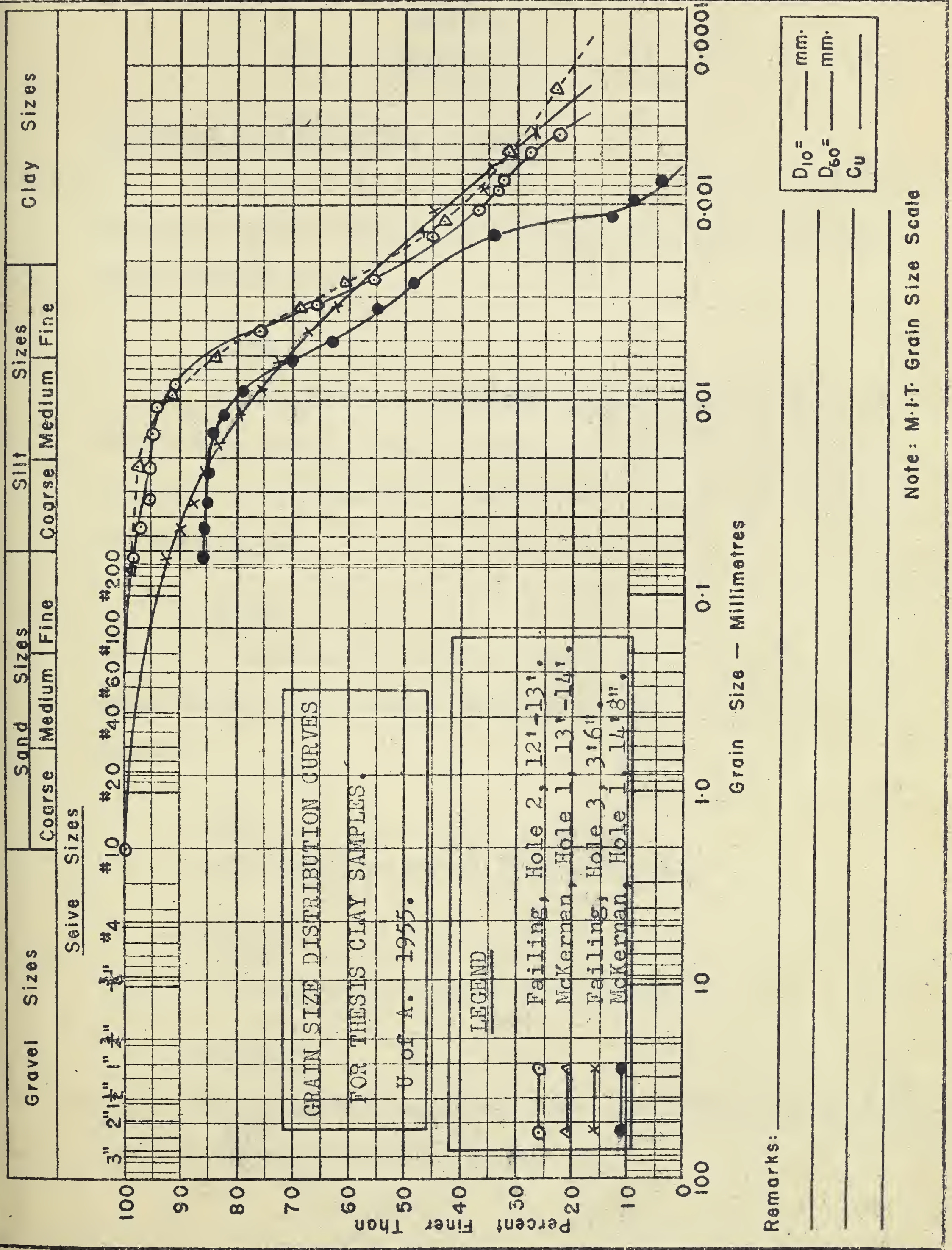
GRAIN SIZE DISTRIBUTION CURVES
FOR FAILING CLAY SAMPLES
U of A April, 1955.

Remarks:
- ⊙—⊙ — Hole 3, 7'–8'
- ×—× — Hole 4, 8'–9'
- *—* — Hole 3, 9'–10'
- △—△ — Hole 3, 5'–6'

PLATE 8.

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
GRAIN SIZE CURVE

PROJECT
SITE
SAMPLE
LOCATION
HOLE DEPTH
TECHNICIAN DATE



Remarks:

Plate 9.

CHAPTER 6

STRENGTH TESTS

Outline of Strength Testing Program

The program was divided into five test series as given below, each with a general aim and each sub-divided into a number of tests with a specific object:

SERIES I

To determine in a qualitative way whether either rapid or slow freezing caused a loss of strength of an highly plastic clay in unconfined compression tests. Conditions for rapid and slow freezing are given below:

Test 1

To determine the validity of using 1 cm. size specimens and Fisher Balance direct reading loading device as opposed to the 1.4 in. specimens.

Test 2

To determine the effects of rapid freezing.

Test 3

To determine the effects of slow freezing.

SERIES II

To determine in a qualitative way whether rapid freezing causes a loss of strength of a silty clay or a shale.

Test 1

Unconfined compression tests on unfrozen and frozen specimens of silty clay, using 1.4 in. specimens.

Test 2

Unconfined and triaxial compression tests on unfrozen and frozen specimens of shale.

SERIES III

To determine whether there is a recovery of strength of an highly plastic clay with time after rapid freezing and thawing, using 1 cm. specimens in unconfined compression tests.

Test 1

Unconfined compression tests after 30 days recovery.

Test 2

Tests after 60 days recovery.

Test 3

Tests after 90 days recovery.

SERIES IV

To determine whether there is a recovery of strength of an highly plastic clay with time following slow freezing and thawing, in unconfined compression tests.

Test 1

Tests after 30 days recovery.

Test 2

Tests after 60 days recovery.

Test 3

Tests after 90 days recovery.

SERIES V

To determine the effects of rapid freezing and thawing on the strength of highly plastic clay in triaxial compression tests.

Preparation and Handling of Specimens:

Specimens for strength tests were pushed from the 3 in. diameter Shelby sample tubes directly into the specimen tubes using an hydraulic testing machine. The specimen tubes had previously been lightly oiled to reduce wall friction and hence undue sample disturbance.

The specimens to be tested unfrozen were removed from the tubes and tested immediately, whereas the corresponding specimens to be frozen were left in the specimen tubes and were placed in sealed metal containers for freezing.

Rapid freezing and thawing of specimens were carried out at temperatures of $-30^{\circ}C$ and $+20^{\circ}C$ respectively, which temperatures were maintained for 24 hours in each case.

Slow freezing and thawing were carried out at temperatures of $-4^{\circ}C$ and $+ 2^{\circ}C$ respectively for 48 hours in each case.

Frozen specimens for immediate testing were removed from the specimen tubes after having beeh thawed and were subjected to compression tests. Specimens which were to be used to check possible recovery of strength with time were left in the sealed containers and were placed in the moist room for testing at a later date.

Test Procedure

Two types of strength test were employed as follows:

(a) Unconfined compression tests using,

  (i) 1 cm. specimens and the Fisher Balance direct reading loading apparatus, for which the detailed test procedure is given as Appendix "A".

  (ii) 1.4 in. specimens and the conventional platform scale loading device, for which detailed test procedure is given as Appendix "B" to this report.

(b) Triaxial compression tests using 1.4 in. specimens, the conventional triaxial cell and platform scale loading device, for which detailed test procedure is given as Appendix "C" to this report.

Data and Results:

SERIES I

Test 1 - Comparison of results using 1.4 in. and 1 cm. specimens. Three samples of Med-Dent clay were used for this test and trials

were run on each. The results are presented on the data sheets and as a plot of moisture content at the end of the test versus logarithm of unconfined compressive strength as obtained using both 1 cm. and 1.4 in. specimens. A separate plot is given for each sample on Plates 10, 11 and 12. The results of all three trials were then plotted on Plate 13 and the best fit line obtained using 1 cm. specimens was transposed to Plate 14. Individual results obtained using 1.4 in. specimens were then plotted on Plate 14 for comparison.

It is evident from Plates 10, 11 and 12 that the 1.4 in. specimens gave consistently lower strengths than the 1 cm. size. The relationship shows up more clearly on Plate 14. It is suggested that this difference may be due to higher capillary pressures in the smaller specimens.

It was decided to continue with the use of both size specimens in Test 2 in order to compare the consistency of this difference before and after freezing, before a decision could be made as to which size of specimen to use for the rest of the program.

UNIVERSITY OF ALBERTA
Dept. of Civil Engineering

FREEZE - THAW EXPERIMENTS

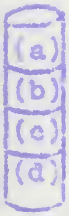
Site: Medical Dental Centre
Location: Jasper Ave., Edmonton.
Hole No. 1 Depth: 2' below ftg.
Moist. Cont. 36.9% Temp. ___ °F
Date sampled: October 54

Test Series: 1 Test No: 1 Trial No: 1 Date of Test: Dec 54

OBJECT: To compare the results obtained using 1 cm. and 1.4 in. specimens in the unconfined compression test.

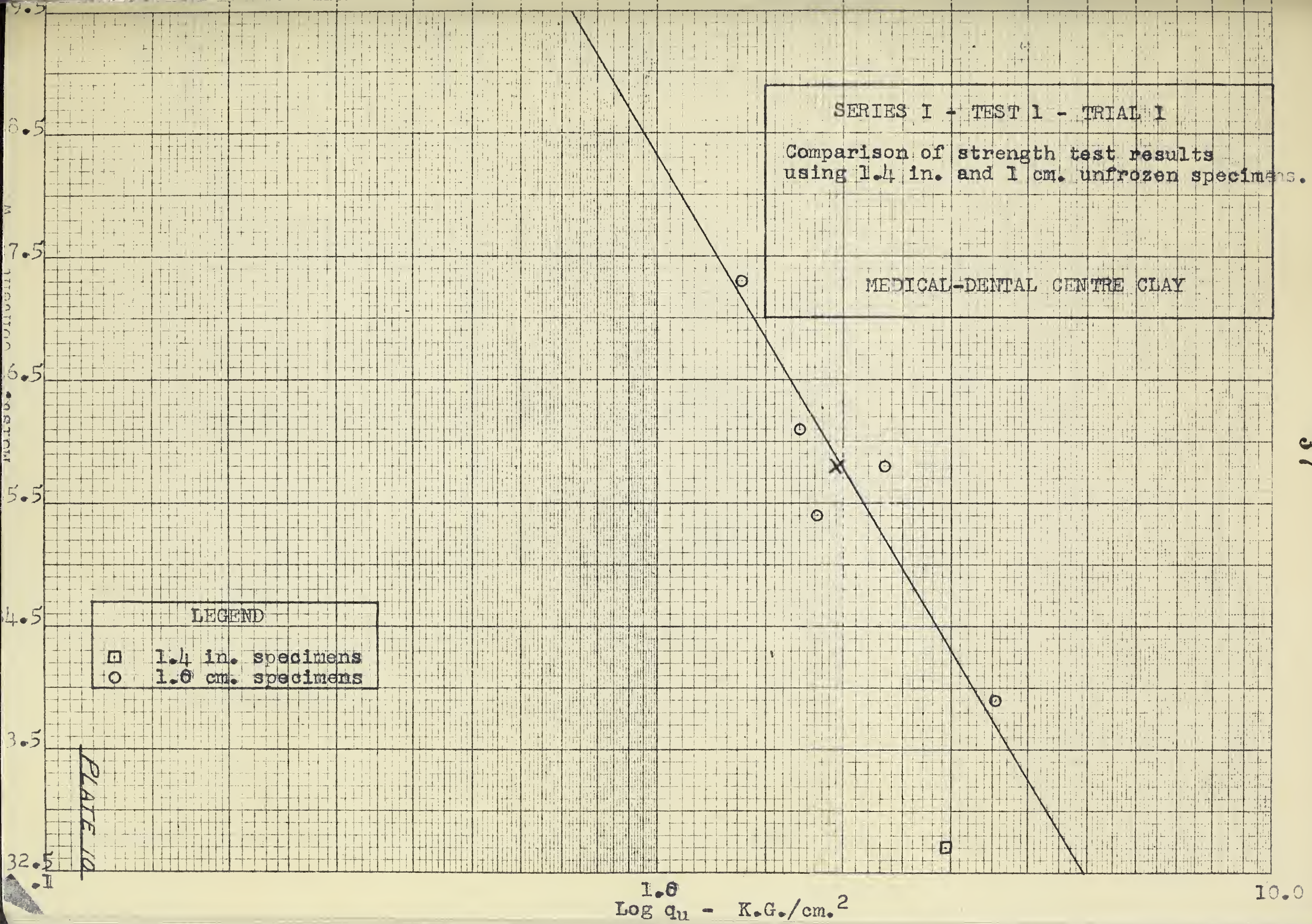
Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.^{2}

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (c)	
			q_u	w%	q_u	w%	q_u	w%	q_u	w%
UNFROZEN  	1 cm.	1	4.50	23.2	2.33	35.8				
		2	3.95	39.2	1.37	37.3				
		3	3.51	33.9	1.7	36.1				
		4			1.81	35.4				
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.		2.94	32.7	1.85	--				
FROZEN at ___ °C for ___ hrs THAWED at ___ °C for ___ hrs Tested after thaw	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS Results for 1.4 in. specimen in layer (b) were discarded, Specimen was loaded too fast.

This soil was medium to highly plastic clay, stiff consistency, W% less than the Wp, occasional layers of silt and pockets of salts.



SERIES I - TEST 1 - TRIAL 1

Comparison of strength test results using 1.4 in. and 1 cm. unfrozen specimens.

MEDICAL-DENTAL CENTRE CLAY

PLATE 10

UNIVERSITY OF ALBERTA Dept. of Civil Engineering FREEZE - THAW EXPERIMENTS	Site: Medical Dental Centre Location: Jasper Avenue, Edmonton. Hole No. 1 Depth: 0.6'below ftg. Moist. Cont. 38.9% Temp. ___ °F Date sampled: October 54

Test Series: 1 Test No: 2 Trial No: 2 Date of Test: Dec 20/54

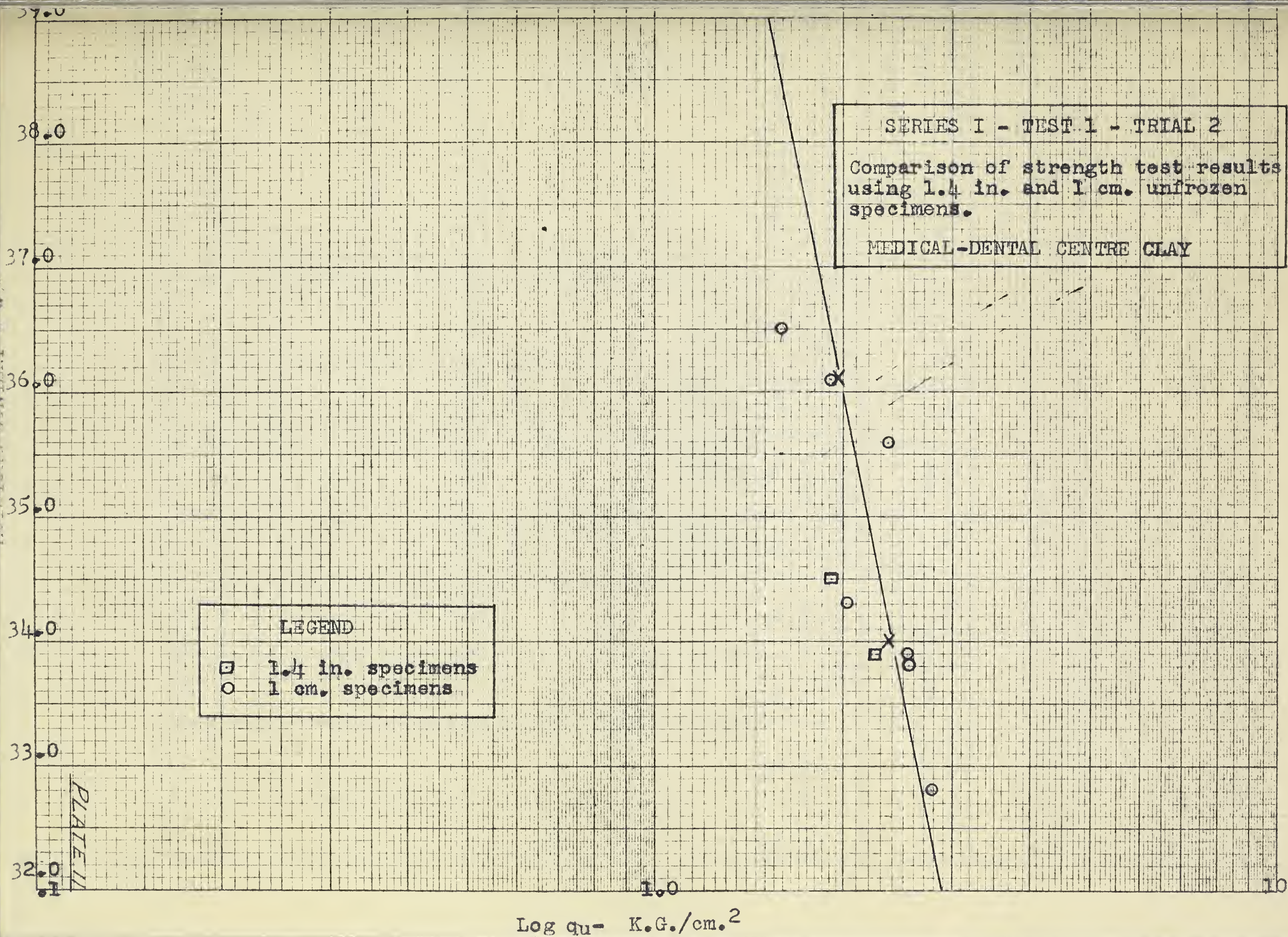
OBJECT: As for Trial 1.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a) q_u	Layer (a) w%	Layer (b) q_u	Layer (b) w%	Layer (c) q_u	Layer (c) w%	Layer (d) q_u	Layer (d) w%
UNFROZEN 	1 cm.	1	2.77	32.8	1.91	36.1				
		2	2.52	33.9	2.37	35.6				
		3	2.02	34.3	1.59	36.5				
		4	2.59	33.8						
		5								
		6								
		7								
		8								
		9								
		10								
		Av.		33.7		36.1				
	1.4 in.		2.28	33.9	1.92	34.5				
FROZEN at ___ °C for ___ hrs THAWED at ___ °C for ___ hrs Tested after thaw	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS


SERIES I - TEST 1 - TRIAL 2

Comparison of strength test results using 1.4 in. and 1 cm. unfrozen specimens.

MEDICAL-DENTAL CENTRE CLAY

PLATE II

UNIVERSITY OF ALBERTA Dept. of Civil Engineering FREEZE - THAW EXPERIMENTS	Site: Medical Dental Centre Location: Jasper Ave., Edmonton. Hole No. 9 Depth: 1'below ftg. Moist. Cont. 34.7% Temp. °F Date sampled: October 54

Test Series: 1 Test No: 1 Trial No: 3 Date of Test: Dec 21/54

OBJECT:

As for Trial 1

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} cm.	U_c	C_c	k cm/sec	Casagrande Class.

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (c)	
			q_u	w%	q_u	w%	q_u	w%	q_u	w%
UNFROZEN	1 cm.	1	1.48	35.6	1.39	34.3				
		2	2.31	33.9	2.88	33.5				
(a) (b) (c) (d)		3	1.36	33.9	3.50	33.3				
		4	2.41	35.1	2.43	31.3				
		5								
		6								
		7								
		8								
		9								
		10								
		Av.		34.6		33.1				
	1.4 in.		1.63	33.6	1.43	30.5				
FROZEN at ___ °C for ___ hrs THAWED at ___ °C for ___ hrs Tested after thaw	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS

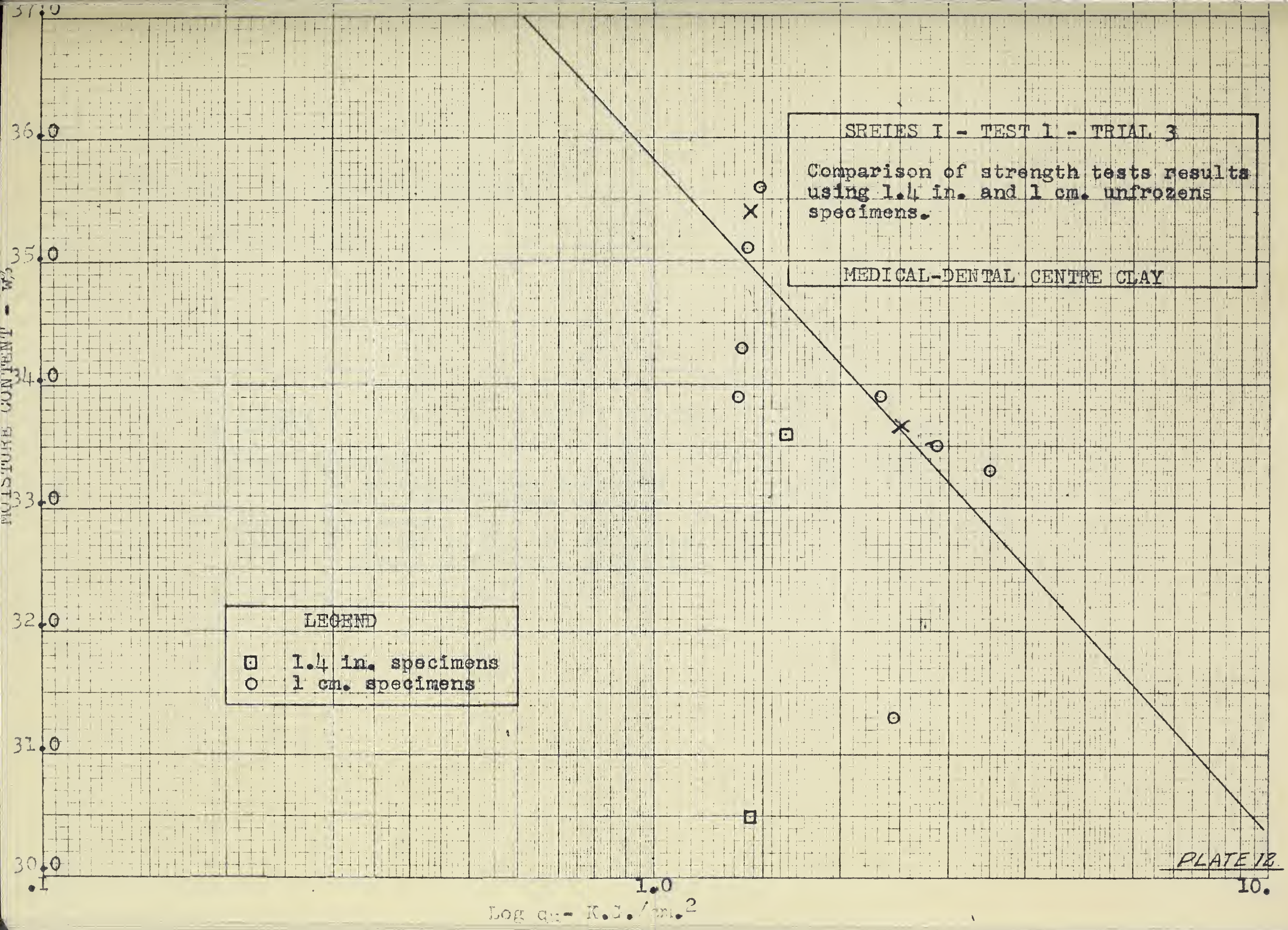

SREIES I - TEST 1 - TRIAL 3
Comparison of strength tests results using 1.4 in. and 1 cm. unfrozens specimens.
MEDICAL-DENTAL CENTRE CLAY
LEGEND
□ 1.4 in. specimens
○ 1 cm. specimens
37.0
36.0
35.0
34.0
33.0
32.0
31.0
30.0
.1
1.0
10.
MOISTURE CONTENT - W%
Log qu - K.G./cm.2

PLATE 12

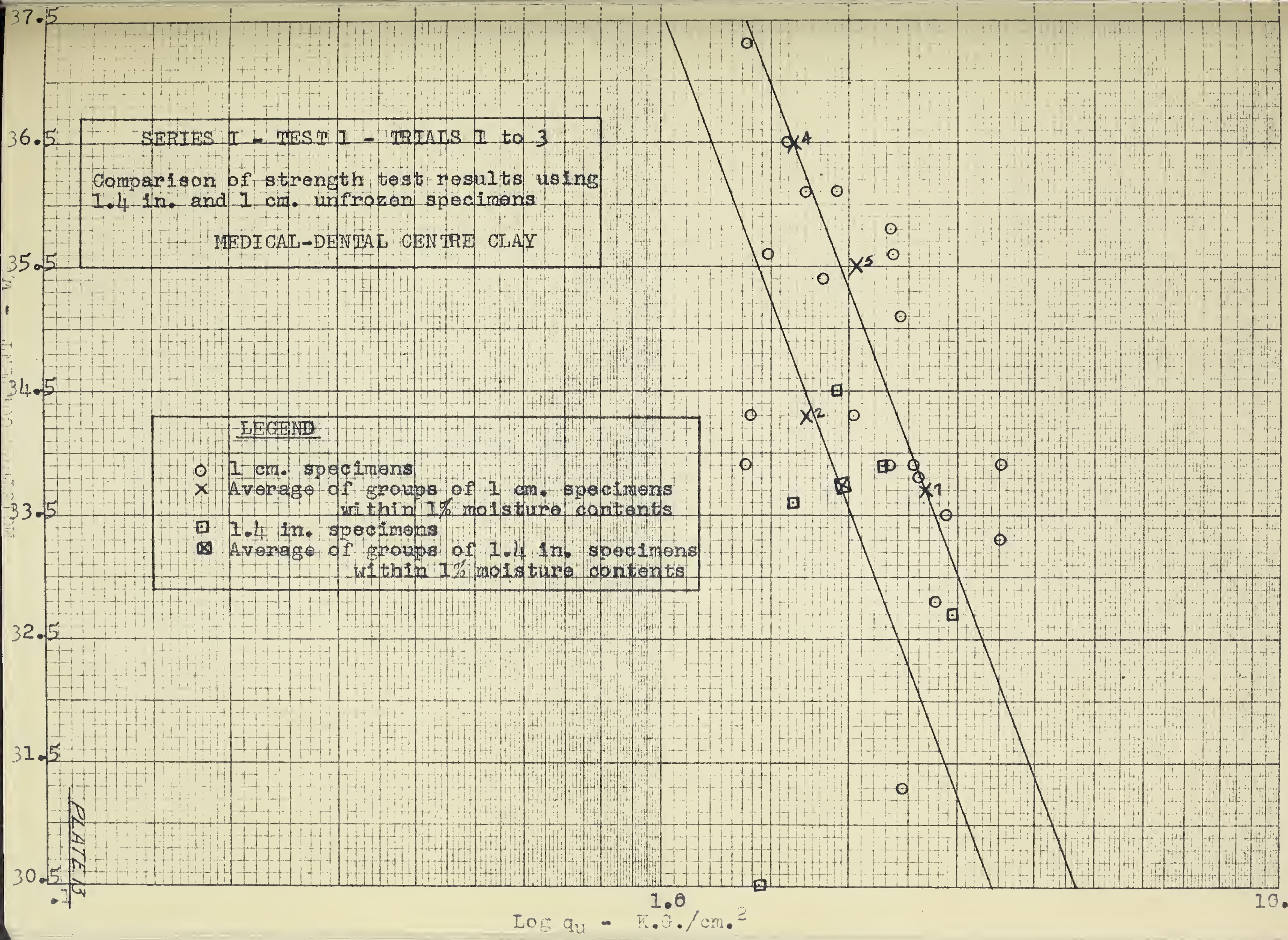

SERIES I - TEST 1 - TRIALS I to 3
Comparison of strength test results using 1.4 in. and 1 cm. unfrozen specimens
MEDICAL-DENTAL CENTRE CLAY
LEGEND
○ 1 cm. specimens
× Average of groups of 1 cm. specimens within 1% moisture contents
⊡ 1.4 in. specimens
⊠ Average of groups of 1.4 in. specimens within 1% moisture contents
37.5
36.5
35.5
34.5
33.5
32.5
31.5
30.5
.1
1.0
10.0
Log qu - K.G./cm.²
1
2
4
5

PLATE 13

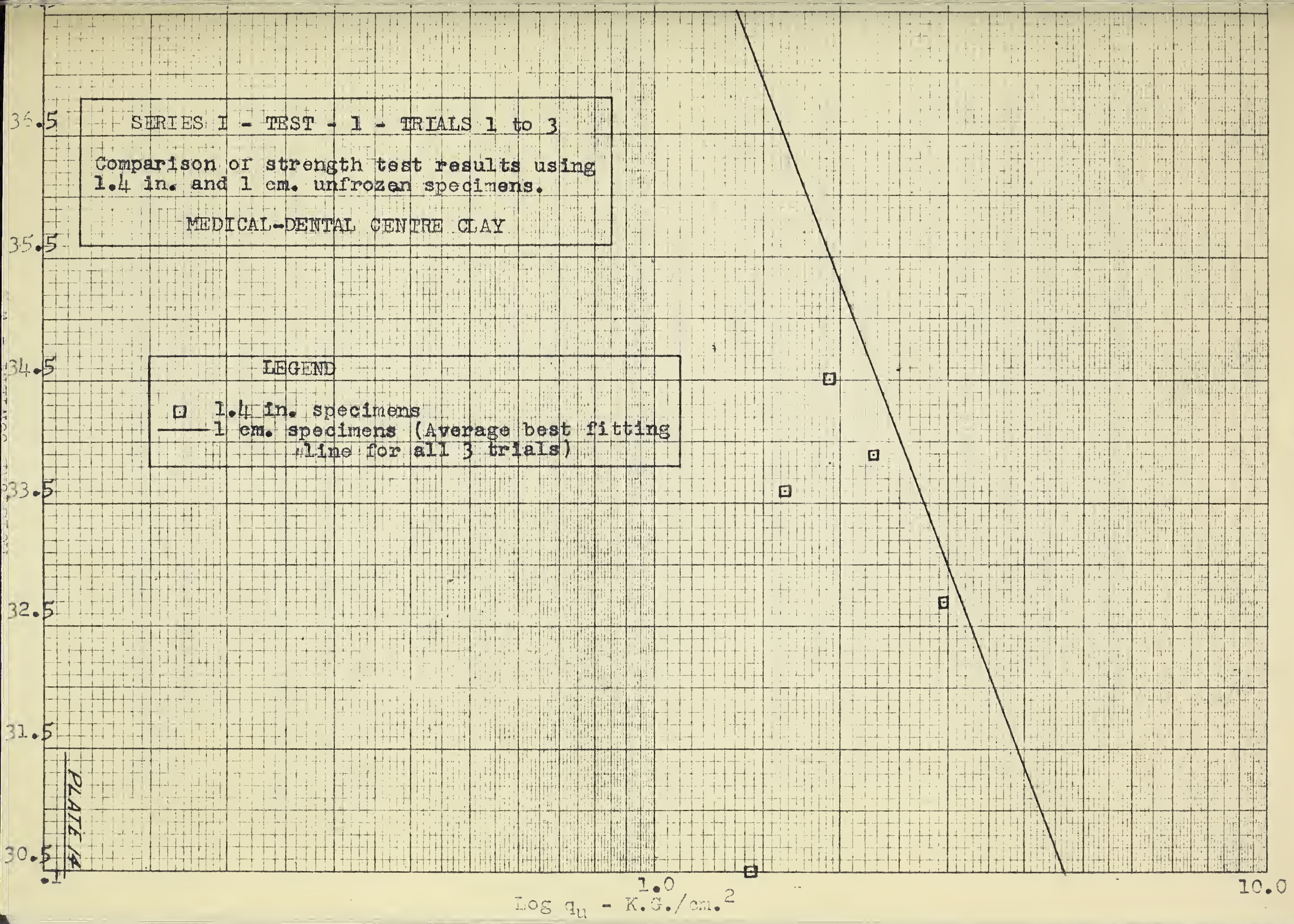

SERIES I - TEST - 1 - TRIALS 1 to 3
Comparison or strength test results using 1.4 in. and 1 cm. unfrozen specimens.
MEDICAL-DENTAL CENTRE CLAY
LEGEND
□ 1.4 in. specimens
—— 1 cm. specimens (Average best fitting line for all 3 trials)
36.5
35.5
34.5
33.5
32.5
31.5
30.5
.1
1.0
10.0
Log q_u - K.G./cm.^{2}

PLATE 14

Test 2 - Tests on unfrozen and rapid frozen specimens.

Trials were carried out on six different samples of Failing clay using both 1 cm. specimens and corresponding 1.4 in. specimens. The results for each trial are presented separately as before on Plates 15 to 20.

Strength values obtained for 1 cm. specimens were averaged within ranges of 1 °/o moisture content for both unfrozen and frozen specimens. The best fitting lines were then constructed through the average points so obtained.

The virgin branch of a consolidation curve for hole 2 at 10.5 to 11.5 ft. was constructed on each plate for comparison. It is pointed out that this curve may not be representative of all samples used, but it has been used as a rough indication of the slope of the w °/o versus log q_u curve to be expected. It has been established by previous investigators, (Ref. 5), that the strength curve runs parallel to the corresponding virgin consolidation curve for a saturated soil.

It will be noted from Plates 15 to 20 that the best fit lines do not appear to run parallel to the consolidation curve, but owing to the relatively small range of moisture contents in each case, there may be insufficient data to establish the lines accurately.

It appears from the above mentioned plates that there is a definite reduction in strength of this soil following rapid freezing

and thawing. Plate 21, on which the results of all trials are plotted, appears to offer a better representation of the results. The general trend of strength curves for all trials is more or less parallel to the consolidation curve.

In view of this fact evidenced from a large number of strength test results as shown on Plate 21, it is considered justifiable to plot the best fit line from a small number of strength test results approximately parallel to the corresponding consolidation curve. This procedure is further substantiated by results from the Vicksburg Triaxial Research study (Ref. 12).

It further appears that the difference between the results using 1.4 in. and 1 cm. specimens is consistent before and after freezing for the same soil. It was therefore decided that, since the object of the whole program was to compare the strength of given soil before and after freezing, the 1 cm. specimens would be used for all subsequent tests. This decision was made on the assumption that the Fisher Balance afforded more accurate stress control and that the greater number of test specimens obtainable from a given quantity of soil would provide a better cross-section of conditions.

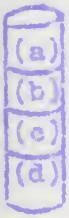
UNIVERSITY OF ALBERTA Dept. of Civil Engineering FREEZE - THAW EXPERIMENTS	Site: Failing Drilling Co. Location: 109St & Kingsway, Edm. Hole No. 2 Depth: 6.0-6.8 ft. Moist. Cont 36.5 % Temp. 46 °F Date sampled: Oct 30th 54

Test Series: 1 Test No: 2 Trial No: 1 Date of Test: Dec 20/54

OBJECT: To determine whether there is a loss of strength suffered by a clay soil as a result of one cycle of rapid freezing and thawing.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.71	73.6	30.5	43.1	.0003	8.6			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.^{2}

Test conditions	Size of Specimen		Layer (a) q_u	Layer (a) w%	Layer (b) q_u	Layer (b) w%	Layer (c) q_u	Layer (c) w%	Layer (c) q_u	Layer (c) w%
UNFROZEN 	1 cm.	1								
		2	.98	38.1						
		3	1.09	37.4						
		4	.79	37.4						
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									
FROZEN at -28 °C for 24 hrs THAWED at +20 °C for 24 hrs Tested immediately after thaw	1 cm.	1								
		2	1.31	34.1						
		3	1.31	35.5						
		4	1.36	35.1						
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.		.65	36.0						

REMARKS Unfrozen specimen No. 3 of Layer (a) discarded
This soil was an highly plastic brownish clay.

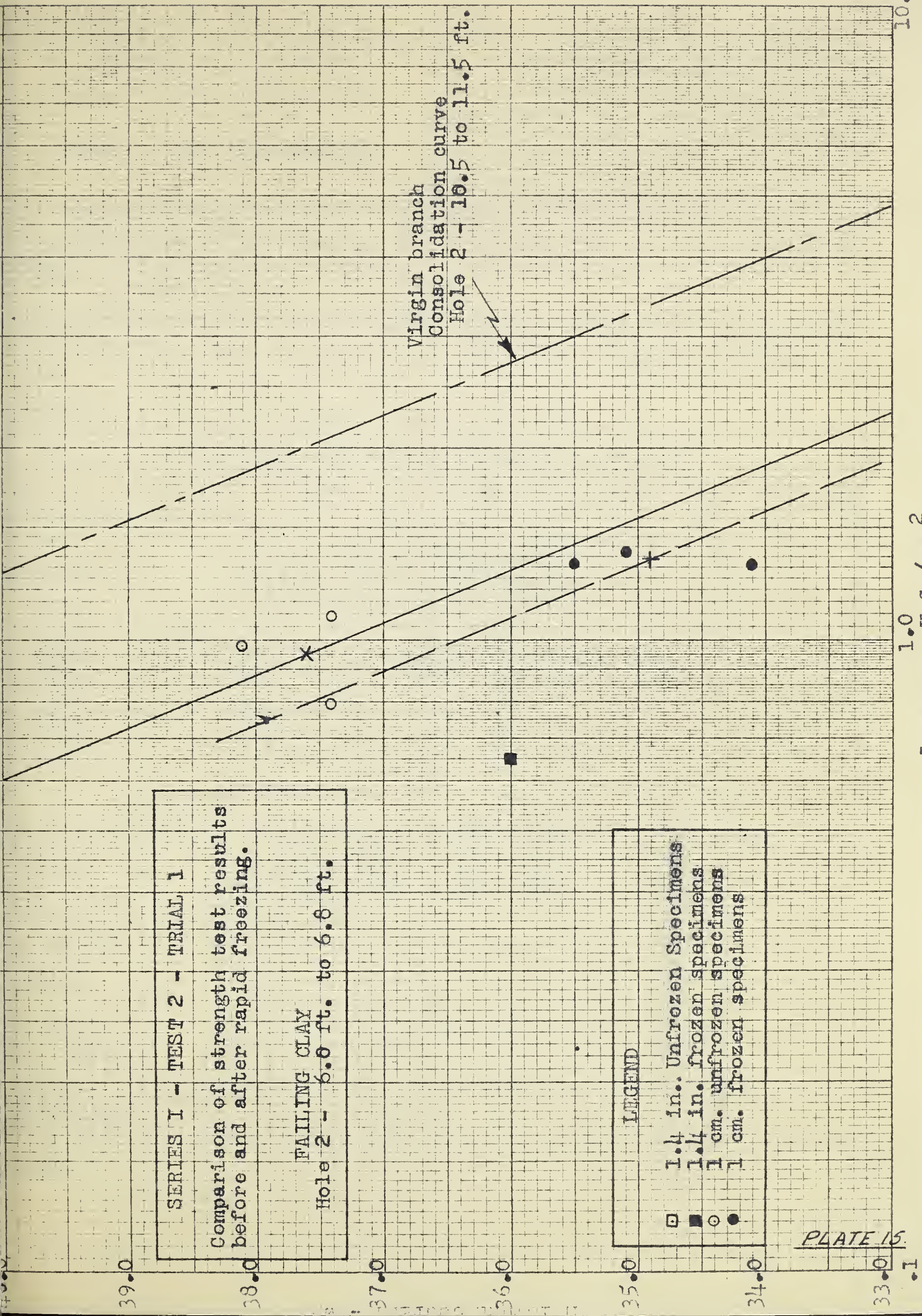

SERIES I - TEST 2 - TRIAL 1
Comparison of strength test results before and after rapid freezing.
FAILING CLAY
Hole 2 - 6.0 ft. to 6.8 ft.
LEGEND
⊡ 1.4 in. Unfrozen Specimens
■ 1.4 in. frozen specimens
○ 1 cm. unfrozen specimens
● 1 cm. frozen specimens
Virgin branch
Consolidation curve
Hole 2 - 10.5 to 11.5 ft.
Log qu - K.G./cm.^{2}
.1
1.0
10.0
39.0
38.0
37.0
36.0
35.0
34.0
33.0

PLATE 15.

UNIVERSITY OF ALBERTA
Dept. of Civil Engineering

FREEZE - THAW EXPERIMENTS

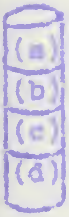
Site: Failing Drilling Co.
Location: 109 St & Kingsway, Edm.
Hole No. 2 Depth: 7.0-8.0 ft.
Moist. Cont. 35.6% Temp. 46°F
Date sampled: Oct 30th 54

Test Series: 1 Test No: 2 Trial No: 2 Date of Test: Dec 27/54

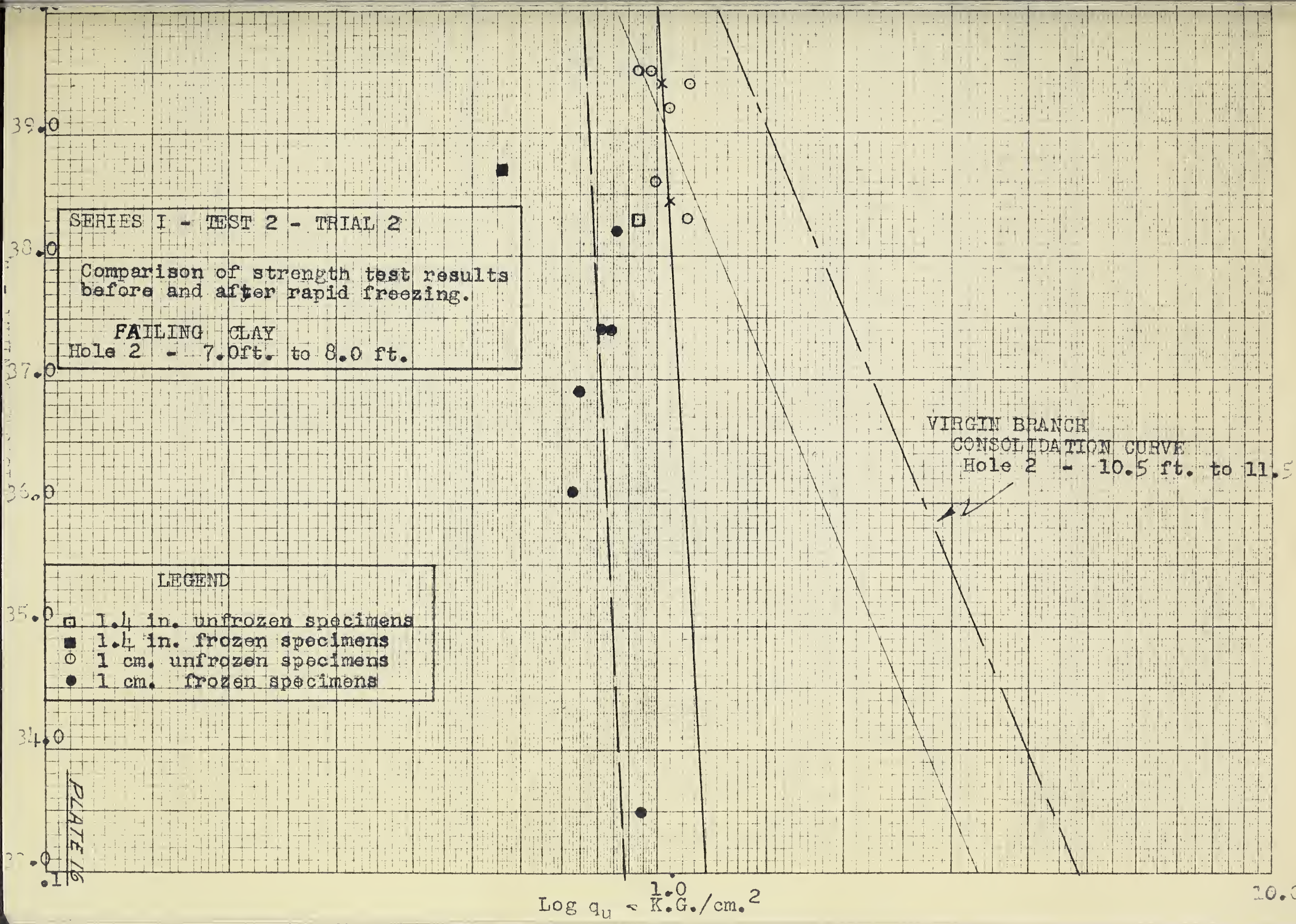
OBJECT: As for Trial 1.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.71	69.0	28.7	40.3	.0003	8.6			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (c)	
			q_u	w%	q_u	w%	q_u	w%	q_u	W
UNFROZEN 	1 cm.	1	.93	39.5	1.13	39.4				
		2	1.12	38.3	.98	39.5				
		3	1.05	39.2	.993	38.6				
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.				.930	38.3				
FROZEN at -28 °C for 24 hrs THAWED at ±20 °C for 24 hrs Tested immediately after thaw	1 cm.	1	.858	38.2	.945	33.5				
		2	.81	37.4	.73	36.1				
		3	.843	37.4	.744	36.9				
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.		.56	38.7						

REMARKS



SERIES I - TEST 2 - TRIAL 2

Comparison of strength test results before and after rapid freezing.

FAILING CLAY
Hole 2 - 7.0ft. to 8.0 ft.

PLATE 16

UNIVERSITY OF ALBERTA Dept. of Civil Engineering FREEZE - THAW EXPERIMENTS	Site: Failing Drilling Co. Location: 109 St & Kingsway Hole No. 3 Depth: 7.0-8.0 ft. Moist. Cont. 38.2% Temp. 46 °F Date sampled: Oct 30th 54

Test Series: 1 Test No: 2 Trial No: 3 Date of Test: Dec 29th-31st/54

OBJECT: As for Trial 1.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.74	69.4	29.0	40.4	.0005	9.1			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.^{2}

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (d)	
			q_u	W%	q_u	W%	q_u	W%	q_u	W%
UNFROZEN 	1 cm.	1			1.20	36.9				
		2			1.41	35.3				
		3			1.26	37.4				
		4			1.45	36.6				
		5			1.60	36.7				
		6			1.45	36.6				
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									
FROZEN at -28 °C for 24 hrs THAWED at +20 °C for 24 hrs Tested immediately after thaw	1 cm.	1			1.23	37.2				
		2	.73	36.2						
		3	.81	35.4						
		4	1.03	36.6						
		5	.99	34.4						
		6	1.21	35.7						
		7	1.09	35.3						
		8								
		9								
		10								
		Av.								
	1.4 in.		.744	36.7						

REMARKS

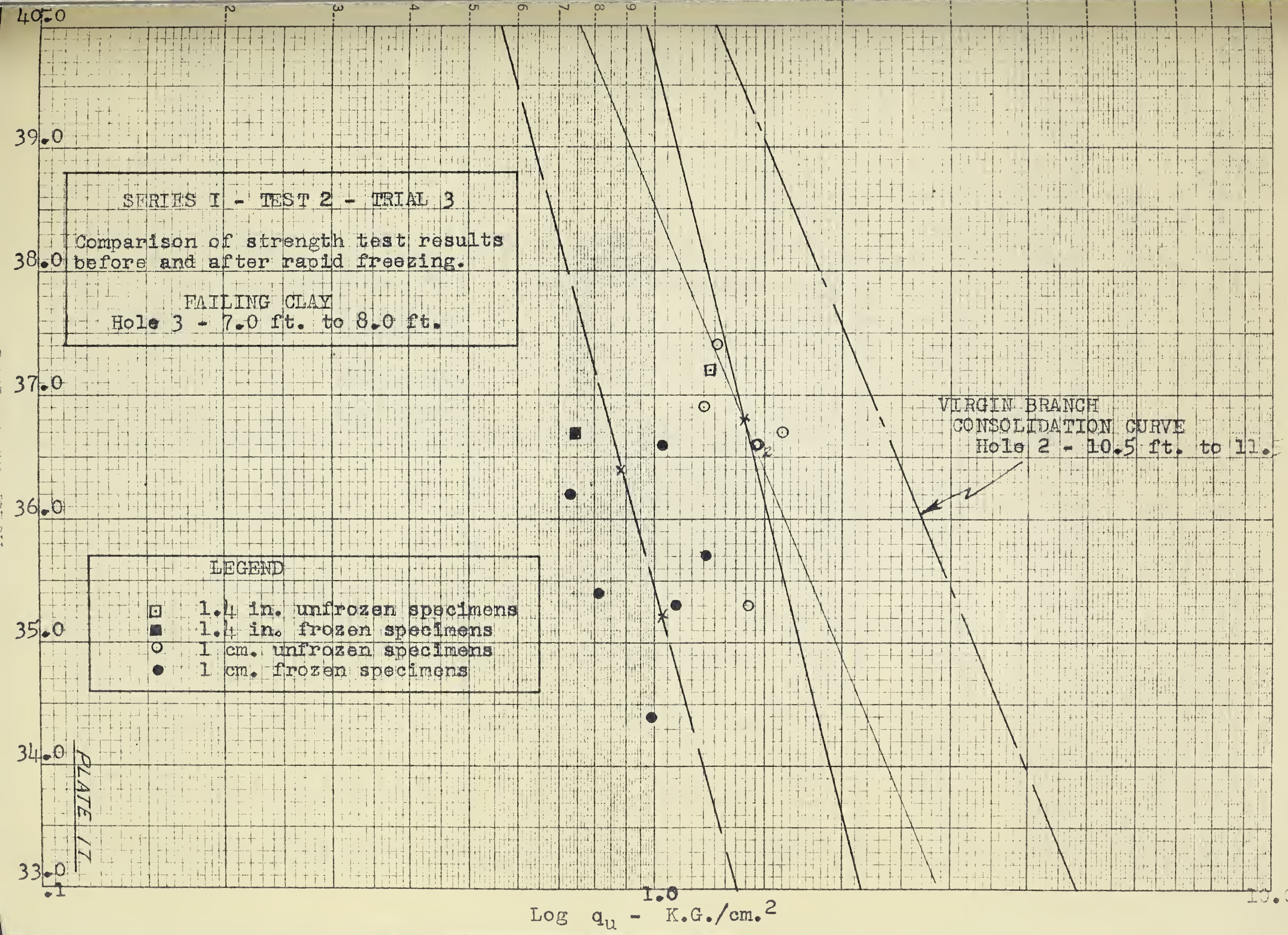

SERIES I - TEST 2 - TRIAL 3
Comparison of strength test results before and after rapid freezing.
FAILING CLAY
Hole 3 - 7.0 ft. to 8.0 ft.
VIRGIN BRANCH CONSOLIDATION CURVE
Hole 2 - 10.5 ft. to 11.5 ft.
LEGEND
⊡ 1.4 in. unfrozen specimens
■ 1.4 in. frozen specimens
○ 1 cm. unfrozen specimens
● 1 cm. frozen specimens
40.0
39.0
38.0
37.0
36.0
35.0
34.0
33.0
.1
2
3
4
5
6
7
8
9
1.0
10.0
2
Log q_u - K.G./cm.^{2}

PLATE 17

UNIVERSITY OF ALBERTA
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FREEZE - THAW EXPERIMENTS

Site: Failing Drilling Co.
Location: 109 St & Kingsway, Edm.
Hole No. 3 Depth: 8.0-9.0 ft.
Moist. Cont. 38.3 Temp. 46°F
Date sampled: Oct 30th 54

est Series: 1 Test No: 2 Trial No: 4 Date of Test: Dec 31/54

EJECT:

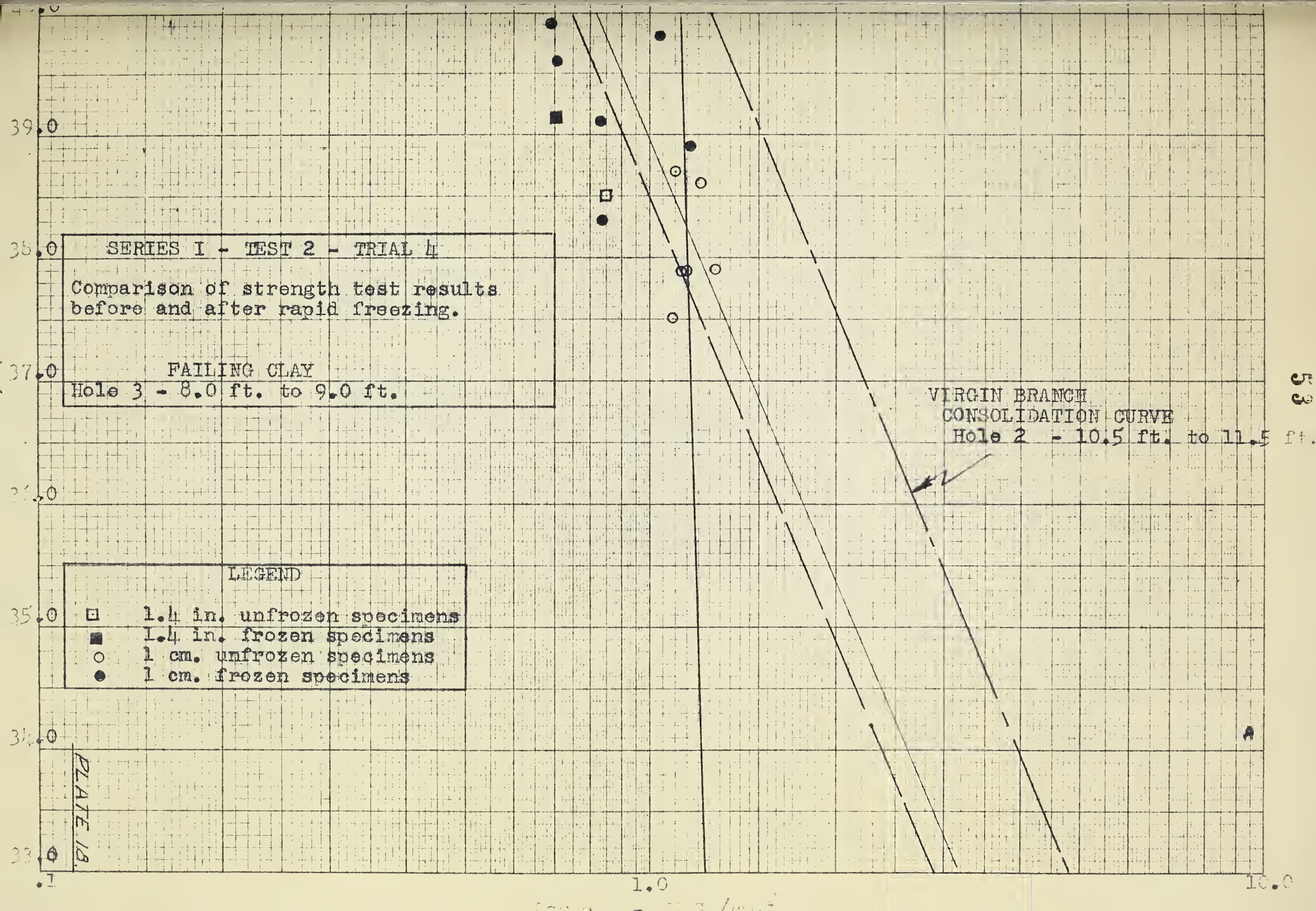
lassification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.74	69.4	29.0	40.4	.0002	17.5			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

est onditions	Size of Specimen		Layer (a) q_u	Layer (a) W%	Layer (b) q_u	Layer (b) W%	Layer (c) q_u	Layer (c) W%	Layer (d) q_u	Layer (d) W%
UNFROZEN 	1 cm.	1			1.28	37.9				
		2			1.10	38.7				
		3			1.09	37.5				
		4			1.145	37.9				
		5			1.205	38.6				
		6			1.135	37.9				
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.				.85	38.5				
FROZEN at -28 °C or 24 hrs THAWED at ±20 °C or 24 hrs Tested mmediately fter thaw	1 cm.	1	.71	39.6						
		2 x	1.03	39.8						
		3	.69	39.9						
		4	.84	38.3						
		5	.83	39.1						
		6 x	1.168	38.9						
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.		.703	39.17						

EMARKS 1. The 1.4 in. frozen specimen showed signs of failure at stress of 0.652 KG/cm sq.

- Unfrozen specimen No. 6, layer (b) is of doubtful value. L nut on Fisher balance slipped imposing a sudden shock load of 850 grams at the 450 gram step. - O.K.
- Frozen specimen Nos. 2 & 6 discarded.



SERIES I - TEST 2 - TRIAL 4

Comparison of strength test results before and after rapid freezing.

FAILING CLAY
Hole 3 - 8.0 ft. to 9.0 ft.

PLATE 18.

UNIVERSITY OF ALBERTA
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FREEZE - THAW EXPERIMENTS

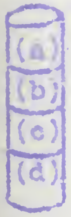
Site: Failing Drilling Co.
Location: 109 St & Kingsway, Edm.
Hole No. 4 Depth: 5.5-6.5 ft.
Moist. Cont. 36.7% Temp. 46°F
Date sampled: Oct 30th 54

Test Series: 1 Test No: 2 Trial No: 5 Date of Test: Dec 31/54

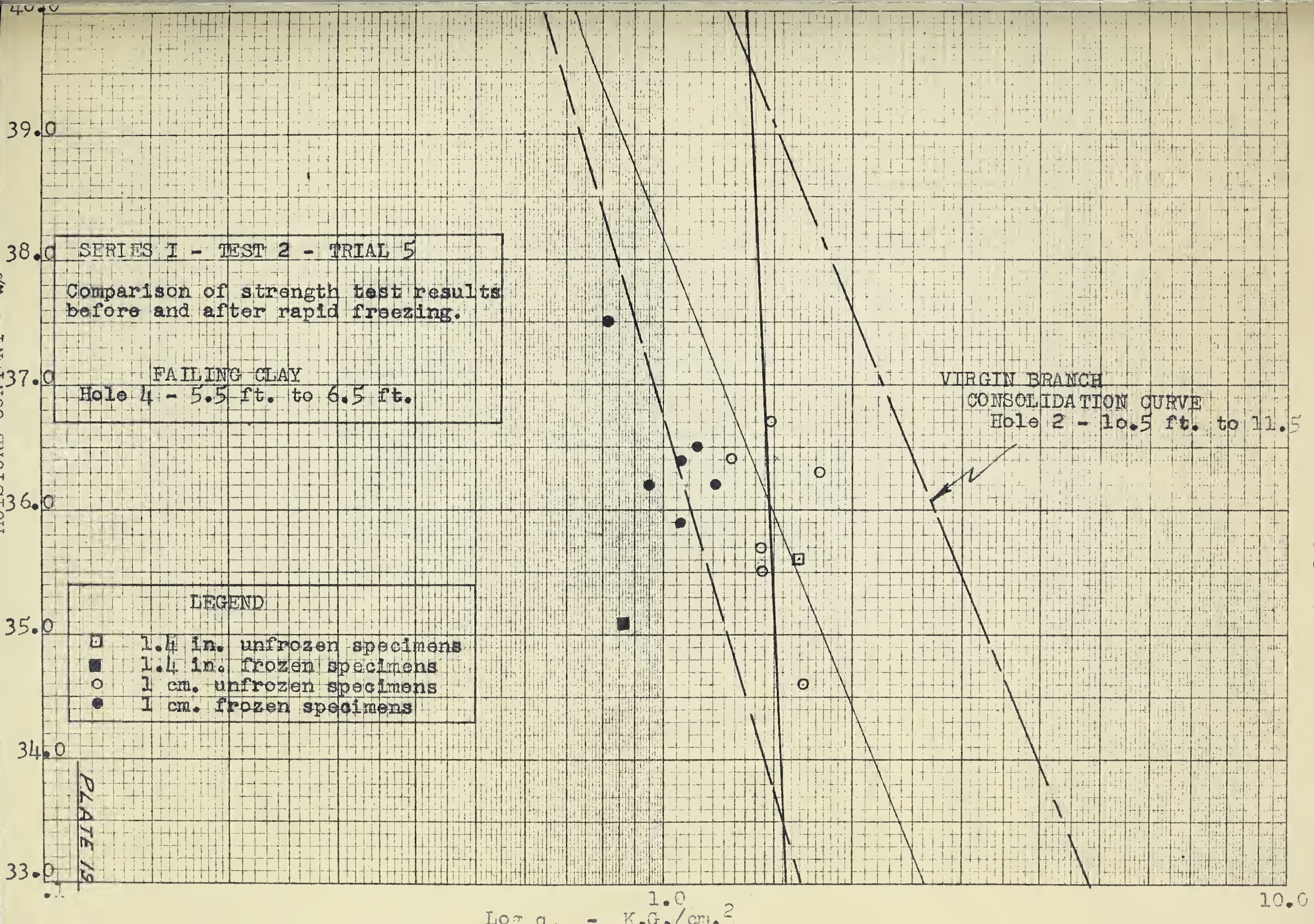
OBJECT: As for trial 1.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.70	87.6	31.8	55.8					CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a) q_u	Layer (a) w%	Layer (b) q_u	Layer (b) w%	Layer (c) q_u	Layer (c) w%	Layer (c) q_u	Layer (c) w%
UNFROZEN 	1 cm.	1			1.43	35.7				
		2			1.44	35.5				
		3			1.48	36.7				
		4			1.67	34.6				
		5		x	1.29	36.4				
		6			1.77	36.3				
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.				1.64	35.6				
FROZEN at -28 °C for 24 hrs THAWED at +20 °C for 24 hrs Tested immediately after thaw	1 cm.	1	1.13	36.5						
		2	1.21	36.2						
		3	.95	36.2						
		4	.81	37.5						
		5	1.07	35.9						
		6	1.07	36.4						
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.		.859	35.1						

REMARKS 1. In testing unfrozen specimen No. 5, layer (b), the machine lurched at the 975 gram load.



SERIES I - TEST 2 - TRIAL 5

Comparison of strength test results before and after rapid freezing.

FAILING CLAY
Hole 4 - 5.5 ft. to 6.5 ft.

PLATE 19

UNIVERSITY OF ALBERTA Dept. of Civil Engineering FREEZE - THAW EXPERIMENTS	Site: Failing Drilling Co. Location: 109 St & Kingsway, Edm. Hole No. 3 Depth: 5 ft. Moist. Cont. 46.9% Temp. 46°F Date sampled: Oct 30th 54

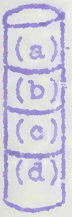
est Series: 1 Test No: 2 Trial No: 6 Date of Test: Jan 11/55

BJECT:

As for trial 1.

lassification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.74	76.2	29.2	46.7	.0005	27.3			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.^{2}

est onditions	Size of Specimen		Layer (a) q_u	Layer (a) W%	Layer (b) q_u	Layer (b) W%	Layer (c) q_u	Layer (c) W%	Layer (c) q_u	Layer (c) W%
UNFROZEN 	1 cm.	1								
		2	3.30	31.65	3.80	28.8				
		3	5.22	29.40	3.83	28.9				
		4	5.50	26.8	6.30	26.4				
		5	4.15	27.95	4.15	28.4				
		6	4.51	29.40	4.16	28.1				
		7	3.50	29.60	3.33	30.6				
		8	3.77	30.00						
		9								
		10								
		Av.								
	1.4 in.									
FROZEN at -32 °C or 24 hrs THAWED at ±20 °C or 24 hrs Tested mmediately fter thaw	1 cm.	1								
		2	1.73	32.8	----	----				
		3	2.15	33.8	4.10	28.7				
		4	2.04	32.7	---	---				
		5	2.13	31.4	10 K.G.	--				
		6	1.95	34.5	4.16	26.5				
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

EMARKS

1. These specimens were dried prior to being tested in an attempt to determine the effects of freezing at lower moisture content.
2. Frozen specimens No. 1, 3 and 4 were dried to such an extent that they could not be broken on the Fisher balance (5 K.G. capacity) and were therefore discarded.

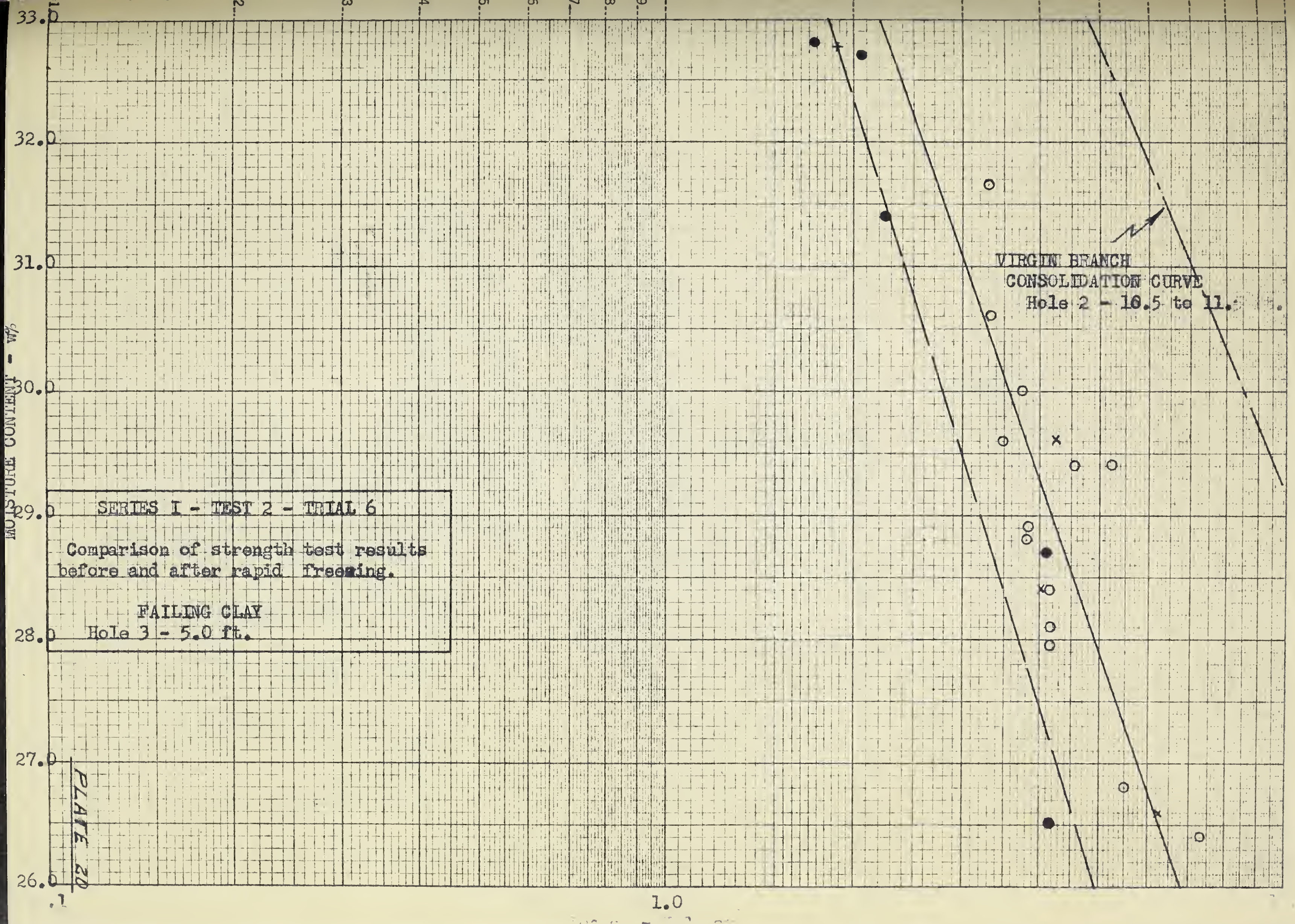
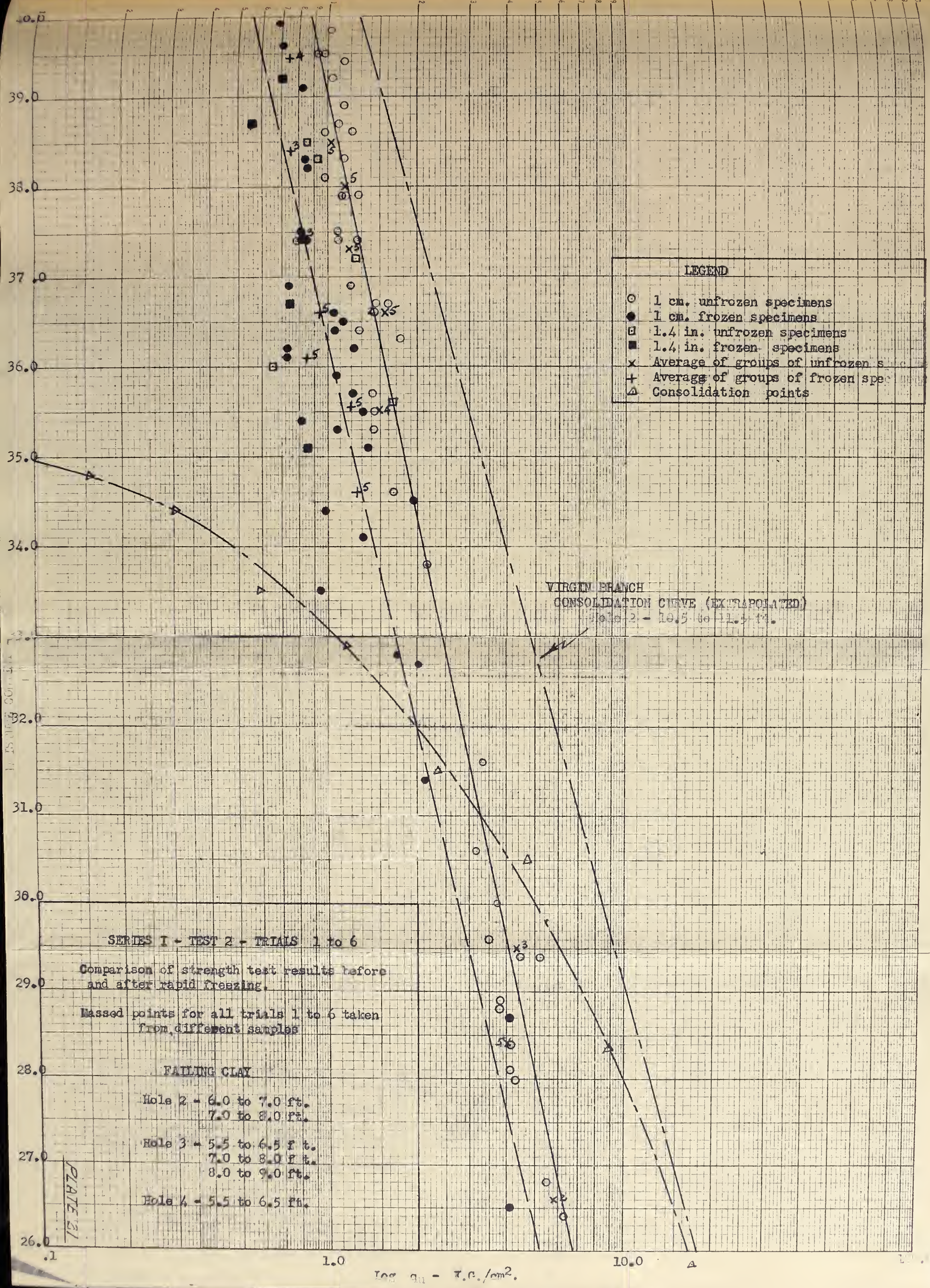
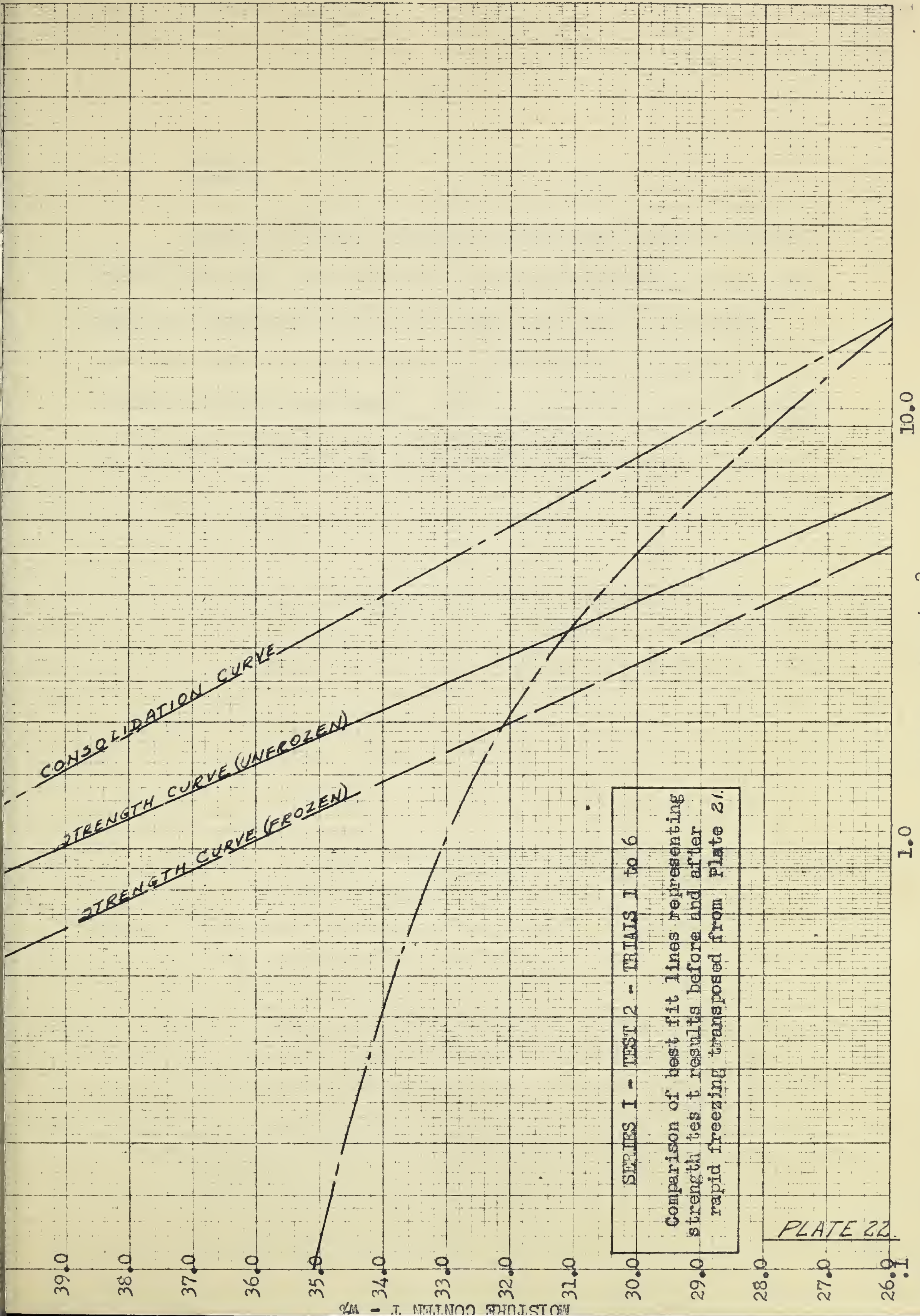

SERIES I - TEST 2 - TRIAL 6
Comparison of strength test results before and after rapid freezing.
FAILING CLAY
Hole 3 - 5.0 ft.
VIRGIN BRANCH
CONSOLIDATION CURVE
Hole 2 - 10.5 to 11.
MOISTURE CONTENT - W%
33.0
32.0
31.0
30.0
29.0
28.0
27.0
26.0
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0

PLATE 20


LEGEND
○ 1 cm. unfrozen specimens
● 1 cm. frozen specimens
⊡ 1.4 in. unfrozen specimens
■ 1.4 in. frozen specimens
× Average of groups of unfrozen s
+ Average of groups of frozen spe
△ Consolidation points
VIRGIN BRANCH
CONSOLIDATION CURVE (EXTRAPOLATED)
Hole 2 - 10.5 to 11.5 ft.
SERIES I - TEST 2 - TRIALS 1 to 6
Comparison of strength test results before and after rapid freezing.
Massed points for all trials 1 to 6 taken from different samples
FAILING CLAY
Hole 2 - 6.0 to 7.0 ft.
7.0 to 8.0 ft.
Hole 3 - 5.5 to 6.5 f t.
7.0 to 8.0 f t.
8.0 to 9.0 ft.
Hole 4 - 5.5 to 6.5 ft.
PLATE 21
40.0
39.0
38.0
37.0
36.0
35.0
34.0
33.0
32.0
31.0
30.0
29.0
28.0
27.0
26.0
.1
1.0
10.0
Log qu - T.c./cm².



SERIES I - TEST 2 - TRIALS I to 6

Comparison of best fit lines representing strength test results before and after rapid freezing transposed from Plate 21.

PLATE 22.

Test 3

Comparison of the unconfined compression strength of highly plastic clay before and after slow freezing is presented in the same manner as before on Plate 23. The best fit line for the frozen specimens again lies well below and to the left of the line representing unfrozen specimens. There is an apparent loss of strength in this soil following slow freezing and thawing.

UNIVERSITY OF ALBERTA
Dept. of Civil Engineering

FREEZE - THAW EXPERIMENTS

Site: Failing Drilling Co.
Location: 109 St & Kingsway
Hole No. 4 Depth: 8.0 ft.
Moist. Cont. 36.3 % Temp. 46 °F
Date sampled: Oct 30th 54

Test Series: 1 Test No: 3 Trial No: 1 Date of Test: Jan 16/55

OBJECT: To determine whether there is a loss of strength of a clay as a result of one cycle of slow freezing and thawing.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.70	69.9	28.4	41.5	.0005	16.6			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.^{2}

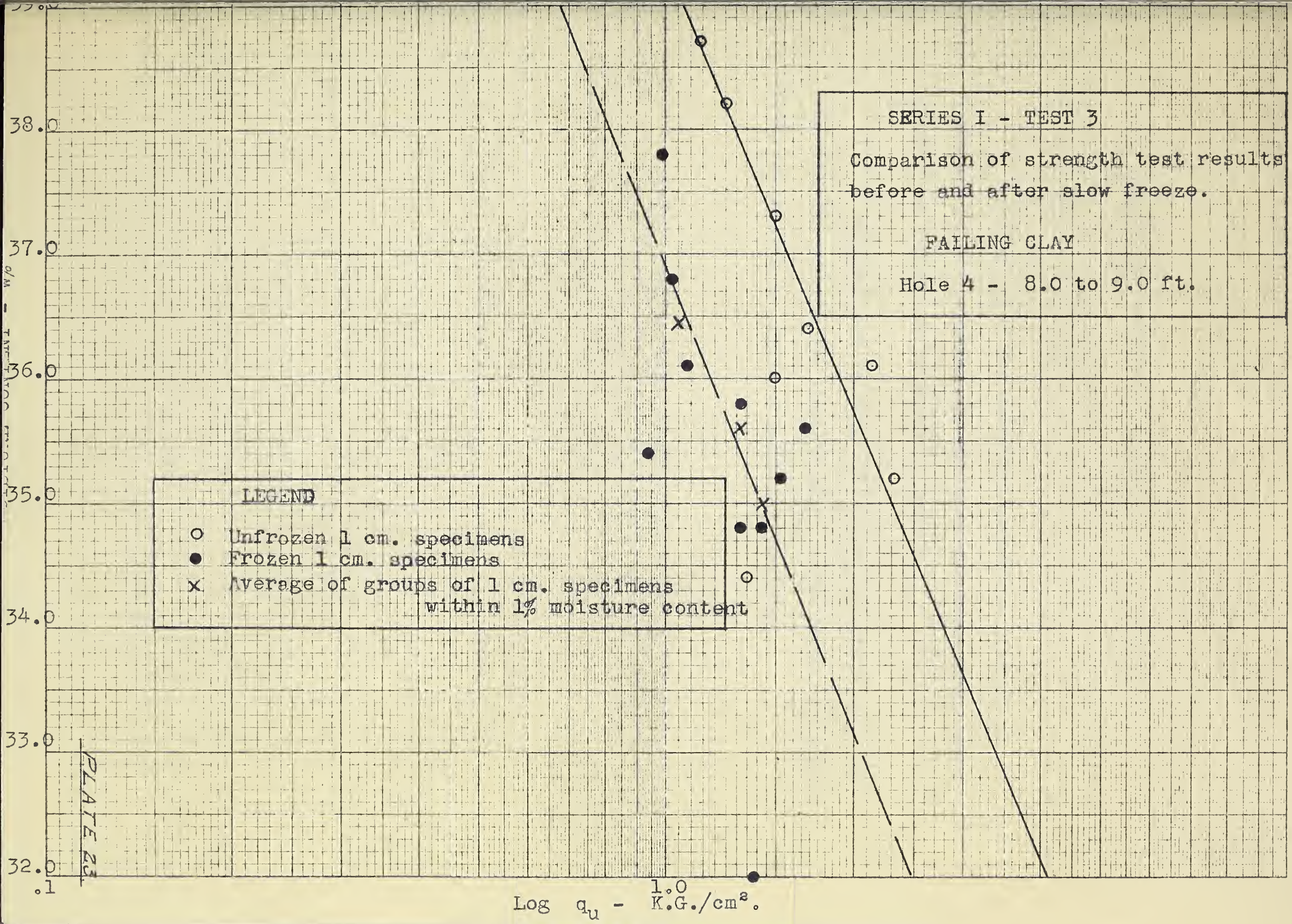
Test conditions	Size of Specimen		Layer (a) q_u	Layer (a) w%	Layer (b) q_u	Layer (b) w%	Layer (c) q_u	Layer (c) w%	Layer (c) q_u	Layer (c) w%
UNFROZEN (a) (b) (c) (d)	1 cm.	1	1.48	32.3	1.69	36.4				
		2	1.35	34.4	1.25	38.2				
		3	1.50	36.0	2.13	36.1				
		4	1.50	26.8	2.31	35.2				
		5	1.50	37.3						
		6	1.14	38.7						
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.						Jan 23/55			
FROZEN at ± 25 °C for 24 hrs THAWED at ± 35 °C for 24 hrs Tested immediately after thaw	1 cm.	1	1.03	36.8	1.32	35.8	1.38	32.0		
		2	0.83	36.0	1.32	34.8	1.67	35.6		
		3	0.71	34.5			.99	37.8		
		4	1.24	32.8			.94	35.4		
		5	1.53	35.2			1.08	36.1		
		6	1.43	34.8						
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS

1. Unfrozen specimens No. 1 and 3 layer (b) showed signs of end restraint.
2. Frozen specimens 2, 4 and 6, layer (a) showed signs of end restraint. No. 6 to be discarded.
3. Unfrozen specimen No. 4, layer (a) and frozen specimen No. 3 layer (b) to be discarded.

NOTE: This test is also control for Series IV, Tests 1, 2 and 3.

4. Frozen specimen No. 2 (layer (c) discarded.


SERIES I - TEST 3

Comparison of strength test results before and after slow freeze.

FAILING CLAY

Hole 4 - 8.0 to 9.0 ft.

PLATE 23.

SERIES II - Silty Clay and Shale

Test 1 - Silty Clay

A comparison of the unconfined compressive strength of McKernan silty clay using 1.4 in. specimens before and after rapid freezing and thawing are presented on Plate 24.

It appeared that freezing and thawing had little effect on this soil; but owing to the limited number of test results available it is not possible to arrive at definite conclusions.

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FREEZE - THAW EXPERIMENTS

Site: McKernan Lake School
Location: Edmonton.
Hole No. 1 Depth 14'8-15'8
Moist. Cont 36.3 % Temp. ___ °F
Date sampled: Nov 6th 54

Test Series: II Test No: 1 Trial No: 1 Date of Test: Feb 19/55

OBJECT: To determine whether there is a loss of strength of a silty clay as a result of one cycle of rapid freezing and thawing.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.73	67.5	20.5	47.0	.001	4			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a) q_u	Layer (a) w%	Layer (b) q_u	Layer (b) w%	Layer (c) q_u	Layer (c) w%	Layer (c) q_u	Layer (c) w%
UNFROZEN (a) (b) (c) (d)	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.		2.03	30.4			2.49	31.9		
FROZEN at -30 °C for 24 hrs THAWED at +20 °C for 24 hrs Tested immediately after thaw	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.				1.69	33.95				

REMARKS 1. Unfrozen specimen layer (c) failed on sand lens.

UNIVERSITY OF ALBERTA
Dept. of Civil Engineering

FREEZE - THAW EXPERIMENTS

Site: McKernan Lake School
Location: Edmonton
Hole No. 1 Depth: 15'9-16'9
Moist. Cont. 34.8% Temp. °F
Date sampled: Nov 6th 54

Test Series: II Test No: 1 Trial No: 2 Date of Test: Feb 19/55

OBJECT: As for Trial 1

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.73	60.5	27.5	33.0					CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (c)	
			q_u	w%	q_u	w%	q_u	w%	q_u	w%
UNFROZEN (a) (b) (c) (d)	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.				1.37	35.95				
FROZEN at -30 °C for 24 hrs THAWED at ±20 °C for 24 hrs	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
Tested immediately after thaw	1.4 in.		1.79	34.9			1.42	34.9		

REMARKS

UNIVERSITY OF ALBERTA
Dept. of Civil Engineering

FREEZE - THAW EXPERIMENTS

Site: McKernan Lake School
Location: Edmonton
Hole No. 1 Depth: 13'4-14'4
Moist. Cont. 35.0% Temp. ___°F
Date sampled: November 6th 54

Test Series: II Test No: 1 Trial No: 3 Date of Test: Feb 19/55

OBJECT:
As for Trial 1.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.73	64.3	17.5	42.2	.0001	23			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.^{2}

Test conditions	Size of Specimen		Layer (a) q_u	Layer (a) w%	Layer (b) q_u	Layer (b) w%	Layer (c) q_u	Layer (c) w%	Layer (c) q_u	Layer (c) w%
UNFROZEN	1 cm.	1								
		2								
		3								
(a)		4								
		5								
(b)		6								
(c)		7								
(d)		8								
		9								
		10								
		Av.								
	1.4 in.		1.25	33.2						
FROZEN	1 cm.	1								
		2								
at ___°C		3								
for ___hrs		4								
		5								
THAWED		6								
		7								
at ___°C		8								
for ___hrs		9								
		10								
Tested		Av.								
after thaw	1.4 in.									

REMARKS

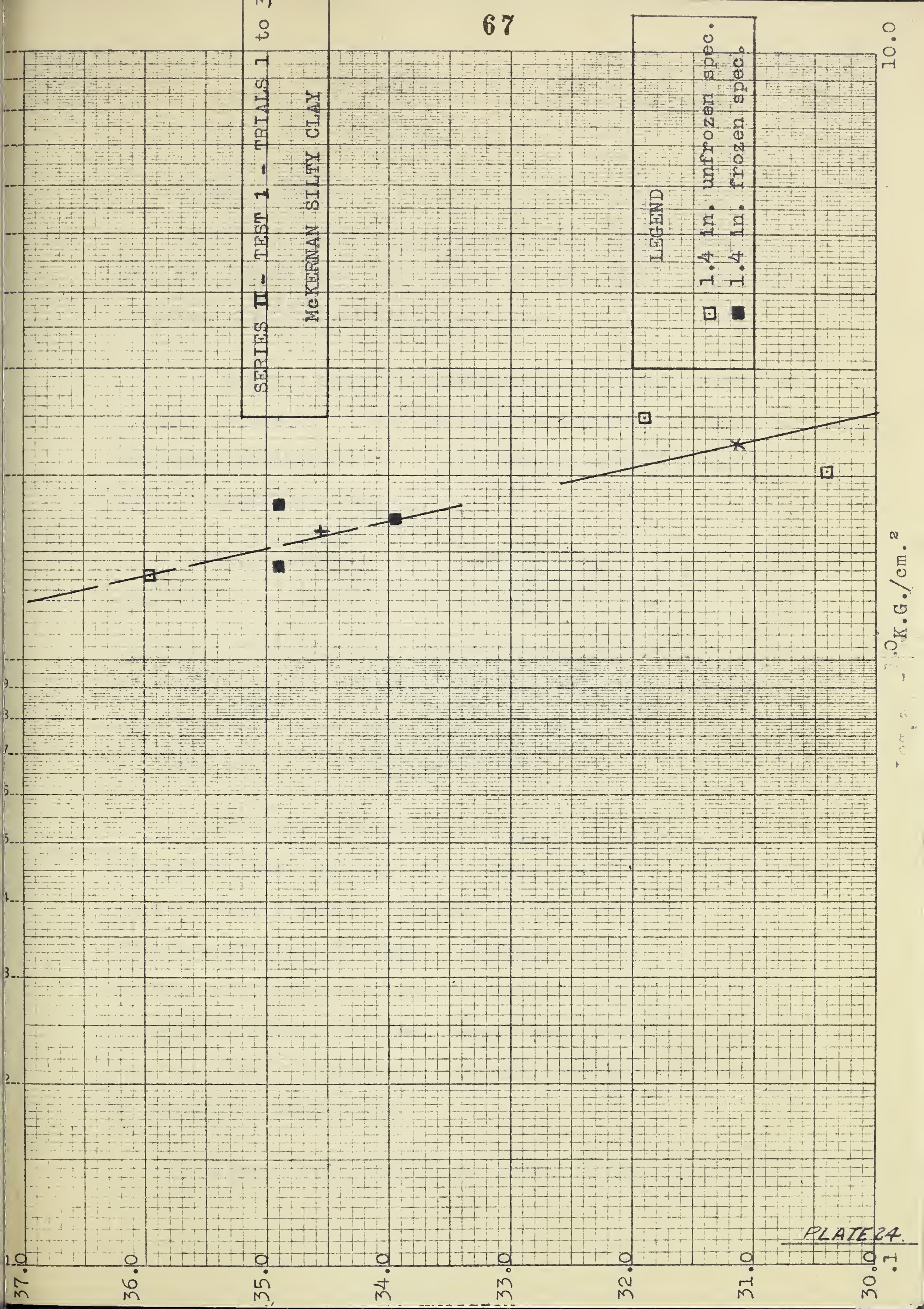

SERIES II - TEST 1 - TRIALS 1 to 3
McKERNAN SILTY CLAY
LEGEND
□ 1.4 in. unfrozen spec.
■ 1.4 in. frozen spec.
K.G./cm.²
37.0
36.0
35.0
34.0
33.0
32.0
31.0
30.0
1.0
.1
10.0
9
8
7
6
5
4
3
2

PLATE 24.

Test 2 - Shale

Comparison of triaxial compression test results for Power House Shale before and after one cycle of rapid freezing and thawing are presented on plates 25 to 28 inclusive.

It had been found in preliminary tests that the material was so badly fissured that it was impossible to obtain an accurate unconfined compressive strength for it. It was therefore decided to carry out triaxial compression tests.

It appeared from a plot of moisture content versus log deviator stress that no relationship could be established between the two within the range of moisture content encountered in these tests. It would therefore appear that the strength of this shale may be independent of moisture content within these limits, and that the soil should probably be treated more as a cohesionless material.

The strength evelope for unfrozen specimens plotted on plate 26 substantiates this opinion in that the soil possesses a relatively high angle of internal friction. The Mohr circles indicate that the soil behaved erratically with respect to moisture content, but, with the exception of one circle, a reasonably well defined strength envelope was obtained having a very small shear strength intercept. This small intercept offers an explanation as to why unconfined compression testing of this soil is impracticable.

The Mohr circles obtained for frozen specimens again show erratic

strength characteristics with respect to water content; but a reasonably well defined strength envelope could be constructed which passed through or near the origin.

The two envelopes are compared on Plate 28. It would appear that the effect of freezing may be to further fracture the soil and to increase the internal friction and interlocking effect of the many more smaller particles. These results are substantiated by extensive tests carried out by the P.F.R.A. on Bearspaw shale which is a similar material. The net result of fracturing due to freezing appears to cause a reduction in the unconfined strength of this shale to practically zero. However, in a confined condition and when no change in moisture content is permitted, there appears to be no reduction in strength of this material as a result of freezing. In fact it would appear from the test results that there may be a slight increase in strength following freezing and thawing where the soil is tested at higher confining pressures, although there is insufficient data to prove this conclusively.

UNIVERSITY OF ALBERTA Dept. of Civil Engineering FREEZE - THAW EXPERIMENTS	Site: City Power House Annex Location: Edmonton Hole No. 1 Depth: Depth of ftg. Moist. Cont. 16.0% Temp. °F Date sampled: December 54

Test Series: II Test No: 2 Trial No: 1 Date of Test: Feb 9/55

OBJECT: To determine whether there is a loss of strength of a shale as a result of one cycle of rapid freezing and thawing.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
							.133		

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.^{2}

Test conditions	Size of Specimen		Layer (a) q_u	Layer (a) W%	Layer (b) q_u	Layer (b) W%	Layer (c) q_u	Layer (c) W%	Layer (d) q_u	Layer (d) W%
UNFROZEN (a) (b) (c) (d)	~~1 cm.~~ XXX	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9	W% =	20.7		17.81		19.55		15.05
		10	p_1 =	9.23		10.32		20.82		12.25
		Av.	p_s =	4.5		2.16		6.0		3.0
	1.4 in.		(p_1-p_3)=	4.73		8.16		14.82		9.25
FROZEN at -30 °C for 24 hrs THAWED at ±20 °C for 24 hrs Tested immediately after thaw	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9	W%=	19.1		14.1		16.18		15.71
		10	p_1=	20.95		40.0		15.61		10.62
		Av.	p_3=	4.0		6.0		6.0		2.0
	1.4 in.		(p_1-p_3)	16.95		34.0		9.61		8.62

REMARKS

1. This soil was a hard gray shale, brittle when dry and badly fissured in its natural state.
2. Attempts to test this soil unconfined failed because of its tendency to break with handling. These tests were therefore triaxial compression tests.

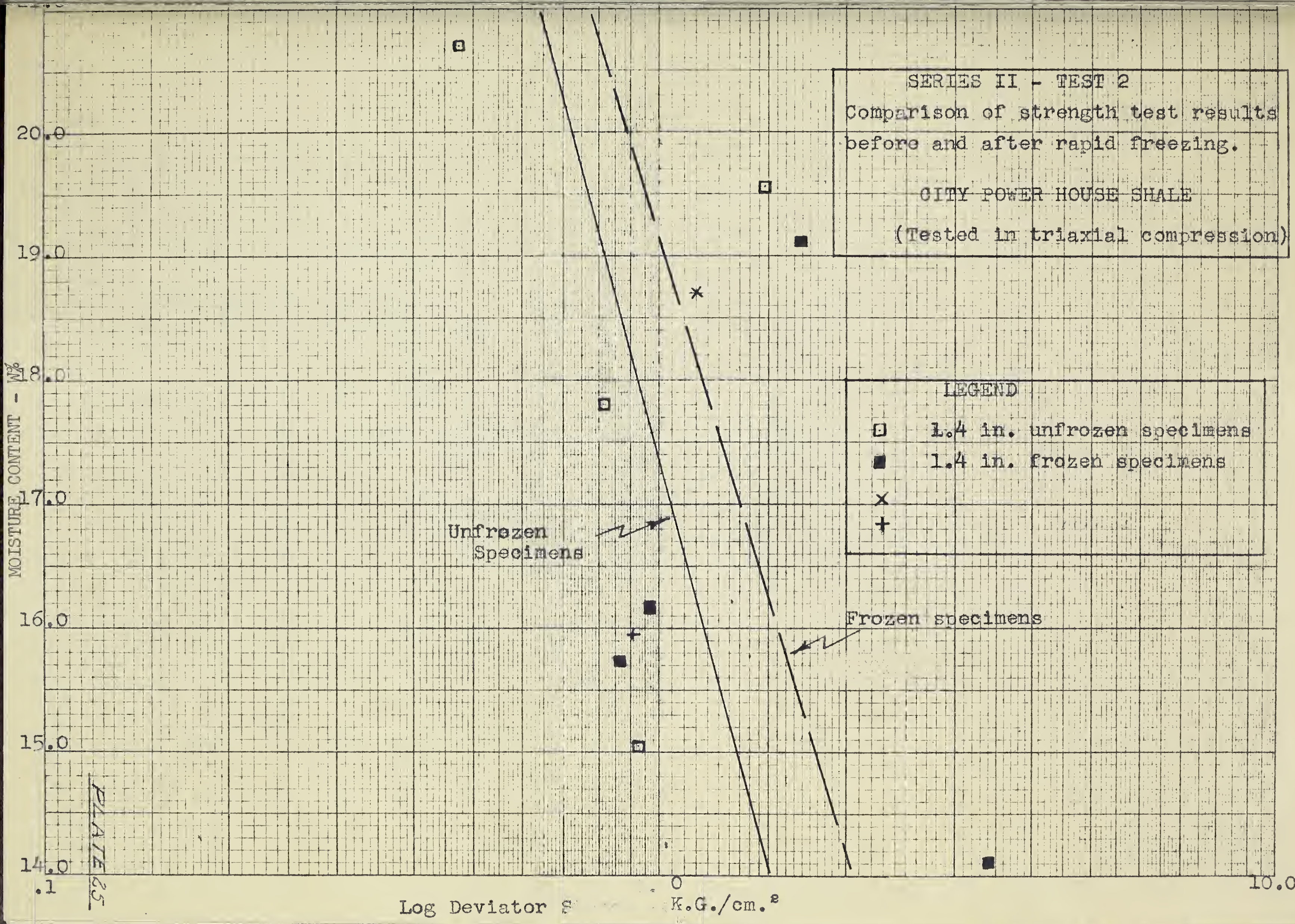
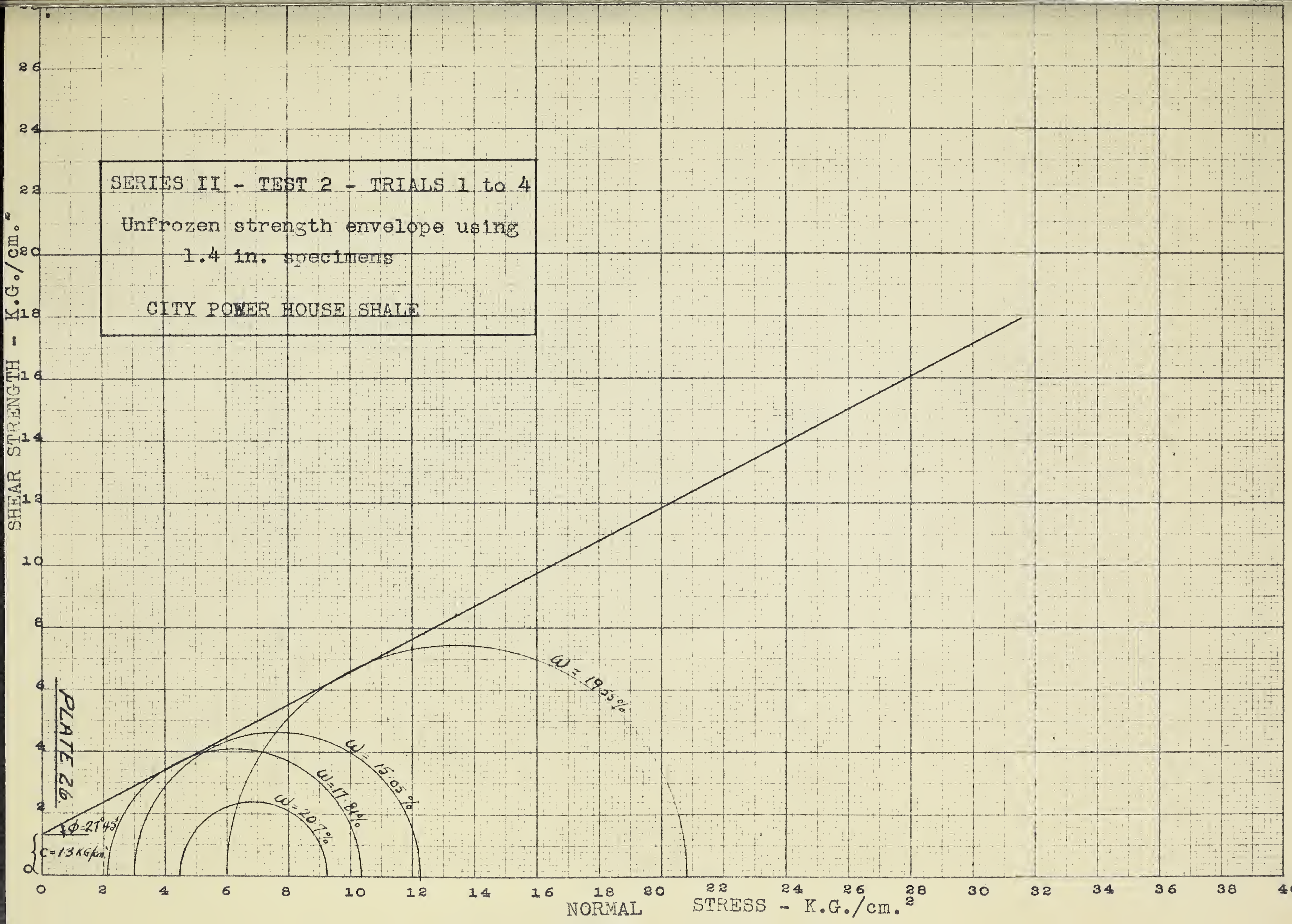

SERIES II - TEST 2
Comparison of strength test results before and after rapid freezing.
CITY POWER HOUSE SHALE
(Tested in triaxial compression)
LEGEND
□ 1.4 in. unfrozen specimens
■ 1.4 in. frozen specimens
×
+
Unfrozen Specimens
Frozen specimens
MOISTURE CONTENT - W%
20.0
19.0
18.0
17.0
16.0
15.0
14.0
.1
0
10.0
Log Deviator S
K.G./cm.²

PLATE 25

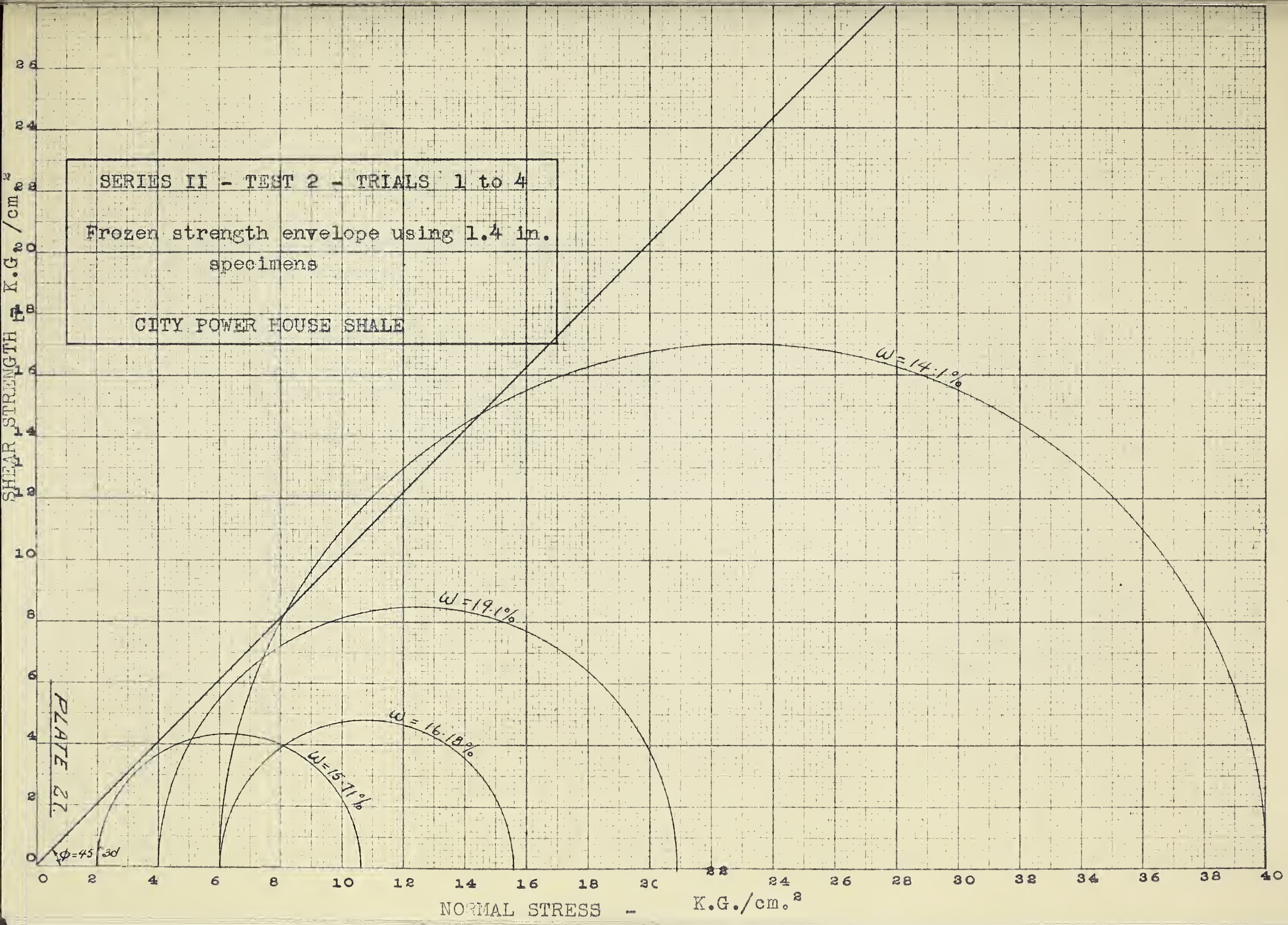


SERIES II - TEST 2 - TRIALS 1 to 4

Unfrozen strength envelope using 1.4 in. specimens

CITY POWER HOUSE SHALE

PLATE 26.



SERIES II - TEST 2 - TRIALS 1 to 4

Frozen strength envelope using 1.4 in. specimens

CITY POWER HOUSE SHALE

PLATE 27.

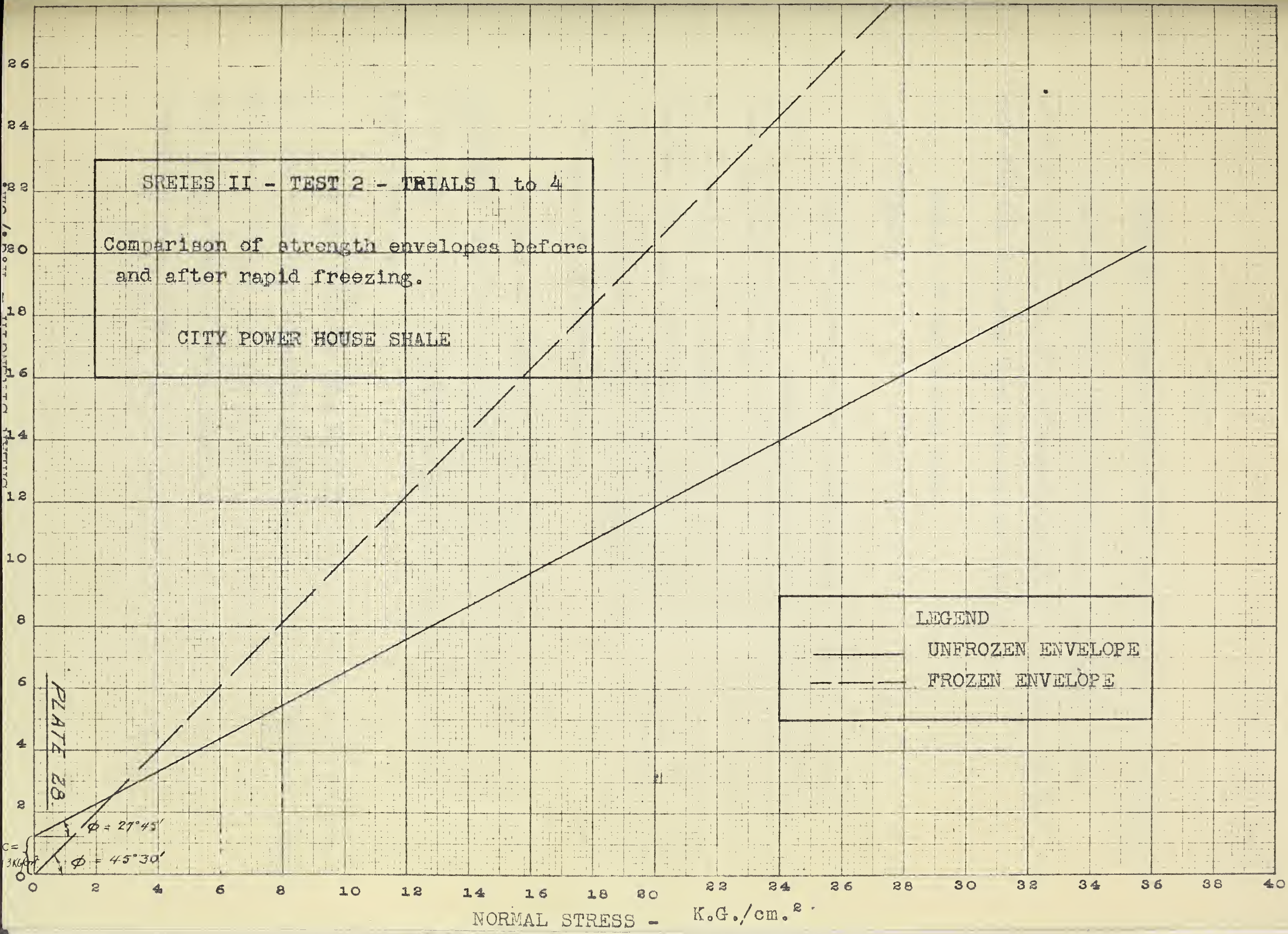

SREIES II - TEST 2 - TRIALS 1 to 4
Comparison of strength envelopes before and after rapid freezing.
CITY POWER HOUSE SHALE
LEGEND
UNFROZEN ENVELOPE
FROZEN ENVELOPE
φ = 27°45′
φ = 45°30′
C = 3KG/cm²
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
NORMAL STRESS - K.G./cm.²

PLATE 28.

SERIES III - Recovery Tests following Rapid Freezing and Thawing.

The object of this series as mentioned previously, was to determine whether there is a recovery of strength of an highly plastic clay with time following rapid freezing and thawing.

Test 1 - 30 day recovery tests

Trial 1

The first of two trials was carried out on Failing clay from hole 2, 12 ft. to 13 ft. Control tests were run on unfrozen and rapid frozen specimens, the results of which are given on the data sheets and as a conventional w^{o}/o vs log q_u plot on Plates 29 and 30. Corresponding specimens were aged for 30 days in sealed containers, after being subjected to rapid freezing and thawing, before being tested. The test results for these specimens are given on Plate 31.

The data was processed in the same manner as for previous tests and best fit lines were constructed for each set of tests. A comparison of the best fit lines for unfrozen, frozen and 30 day old specimens is given on Plate 32.

In processing the data for these tests, it appeared that the best fit lines would have a break in the vicinity of 32 to $34^{o}/o$ moisture content. This was considered reasonable in view of the findings of the Triaxial Shear Report, (Ref. 5), which reports such an occurrence in the case of unsaturated soils. However, in view of the relatively short range of

moisture contents encountered in this test series, it was considered inadvisable to establish the slope of the curves by the trend of the points alone. There was considerable scatter of the points, but from a cursory inspection it appeared that the scatter was predominantly in the vertical direction. By that is meant that there often occurred a group of points with very similar strength values but with widely scattered moisture contents. If the points alone were used to establish the slope of the curve, it was found that in most cases the curve would be almost vertical; or that if two or more groups appeared, each with an apparently genuine difference in average moisture content, two or more strength curves could be constructed through them. If these separate curves were projected, they would in some cases intersect giving what appeared to be a bent or broken line.

It is felt there may be one or both of two reasons for this vertical scatter. It could be due to slight discrepancies in moisture content determinations, or it may be the result of variations in the degree of saturation of different specimens.

It has been shown by Taylor, (Ref. 7). that the presence of air in a soil results in a lower strength, particularly as determined in unconfined compression tests, than would be expected for the given moisture content.

The foregoing leads one to suspect that there may be a family

[illegible]

[illegible]

[illegible]

[illegible]

of strength curves running approximately parallel to the virgin consolidation curve, each for a given degree of saturation.

It was therefore decided to locate the strength curves with respect to the greatest concentration of experimental points, but to establish the slope by the trend of the consolidation curve. It is considered that the best fit line so obtained probably represents the average degree of saturation of all specimens used for the trial.

It is conceded that, when comparing the best fit lines for unfrozen, frozen, etc., specimens, the lines may represent a slightly different degree of saturation in each case and therefore be at variance because of this difference. However, only one sample tube has been used for any one test so that it is reasonable to expect a similar average degree of saturation for groups of specimens taken from a given sample.

On this basis it was possible to compare, at least qualitatively, the strength of the soil before freezing, immediately after freezing and thawing, and 30 days after thawing as shown on Plate 32.

It appears that the loss of strength as a result of freezing varies with moisture content, but by taking 35°/o as the average moisture content, the reduction in strength at that point was of the order of 14°/o.

It was found that the soil did recover strength after aging 30 days. This recovery amounted to approximately 75°/o of the total loss for this particular sample.

UNIVERSITY OF ALBERTA Dept. of Civil Engineering FREEZE - THAW EXPERIMENTS	Site: Failing Drilling Co. Location: 109 St & Kingsway, Edm Hole No. 2 Depth: 11 ft. Moist. Cont. 37.8% Temp. 46°F Date sampled: October 30 54

Test Series: III Test No: 1 Trial No: 1 Date of Test: Jan 11/55

OBJECT: To determine whether there is a recovery of strength with time following one cycle of rapid freezing and thawing.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.71	70.7	28.0	42.7	.0002	15.0			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (c)	
			q_u	w%	q_u	w%	q_u	w%	q_u	w%
UNFROZEN (a) (b) (c) (d)	1 cm.	1	1.37	38.4	4.07	31.5	3.37	34.0		
		2	1.69	39.3	5.20	29.1	3.85	34.95		
		3	1.25	35.9	4.46	32.7	1.60	38.9		
		4	1.55	38.7	3.78	31.5	4.10	35.3		
		5	1.37	37.4	3.40	31.1	2.60	33.9		
		6	1.35	40.7	3.43	33.3	3.12	33.1		
		7	1.40	37.8	5.95	32.8	4.60	31.05		
		8	1.37	38.4	3.37	34.8				
		9	1.35	38.9	3.14	34.8				
		10	1.74	35.5	3.64	35.9				
		Av.			3.37	34.6				
	1.4 in.									
FROZEN at -32 °C for 24 hrs THAWED at ±20 °C for 24 hrs Tested immediately after thaw	1 cm.	1	1.87	35.5	3.07	36.3	3.97	31.0		
		2	1.33	36.1	2.36	36.8	3.87	32.7	(discarded)	
		3	1.39	38.6	2.64	36.1	4.75	32.7	(discarded)	
		4	1.41	36.4	3.53	32.7	3.12	31.5		
		5	1.20	36.9	2.83	33.1	3.65	32.3		
		6			3.50	32.9	4.28	30.0		
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS

1. This is control data for the 30 day recovery tests given on next page.
2. Specimen 2, layer (a) unfrozen failed during change over of load from 500 to 600 grams and was discarded.
3. Evidence of end restraint in following:

Unfrozen - layer (a) No. 6: Layer (b) Nos. 7 (discarded) and 11:
layer (c) No. 5
Frozen - Layer (a) No. 1 and 4: Layer (c) No. 2

UNIVERSITY OF ALBERTA Dept. of Civil Engineering FREEZE - THAW EXPERIMENTS	Site: Failing Drilling Co. Location: 109 St & Kingsway Hole No. 2 Depth: 11 ft. Moist. Cont. 37.8% Temp. 46°F Date sampled: Oct 30th 54

Test Series: III Test No: 1 Trial No: 1 Cont'd Date of Test: Feb 14/55

OBJECT: To determine whether there is a recovery of strength of a clay after 30 days following one cycle of rapid freezing and thawing.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.71	70.7	28.0	42.7	.0002	15.0			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.^{2}

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (c)	
			q_u	W%	q_u	W%	q_u	W%	q_u	W%
UNFROZEN (a) (b) (c) (d)	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									
FROZEN at -32 °C for 24 hrs THAWED at ± 20 °C for 24 hrs Tested 30 days after thaw	1 cm.	1	4.46	31.8	2.18	35.2	3.07	35.95		
		2	3.98	33.7	1.57	35.5	2.18	34.8		
		3	4.72	31.6	1.44	38.6	3.50	33.15		
		4	2.34	37.4	1.09	38.5	2.50	34.6		
		5	2.33	37.0	1.27	36.5	3.84	34.1		
		6	2.58	37.4			3.20	34.4		
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS 1. Evidence of end restraint in specimens No. 1 (discarded) and 2 (O.K), Layer (c)

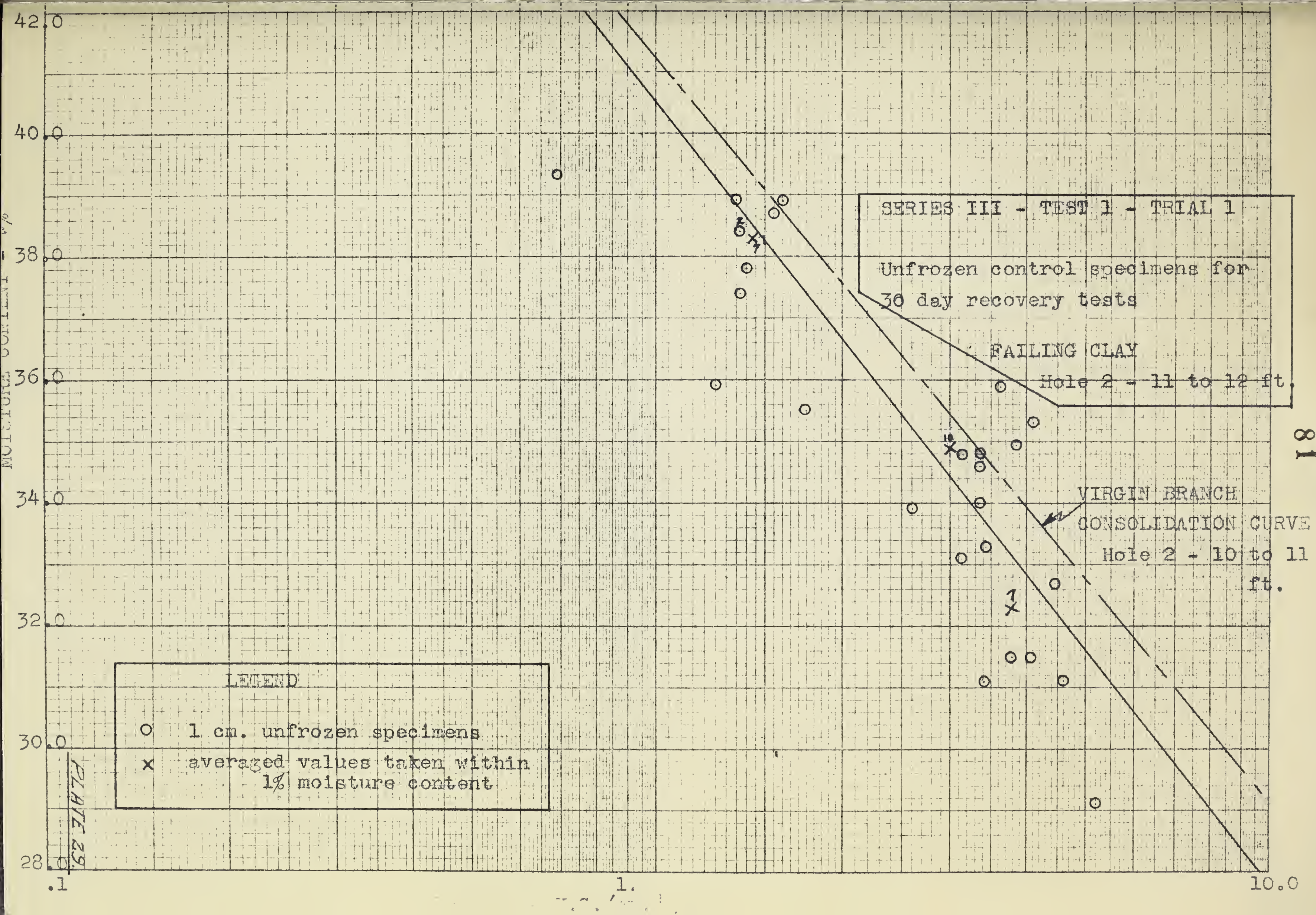

42.0
40.0
38.0
36.0
34.0
32.0
30.0
28.0
.1
1.
10.0
SERIES III - TEST 1 - TRIAL 1
Unfrozen control specimens for
30 day recovery tests
FAILING CLAY
Hole 2 - 11 to 12 ft.
VIRGIN BRANCH
CONSOLIDATION CURVE
Hole 2 - 10 to 11
ft.
LEGEND
○ 1 cm. unfrozen specimens
× averaged values taken within
1% moisture content

PLATE 29.

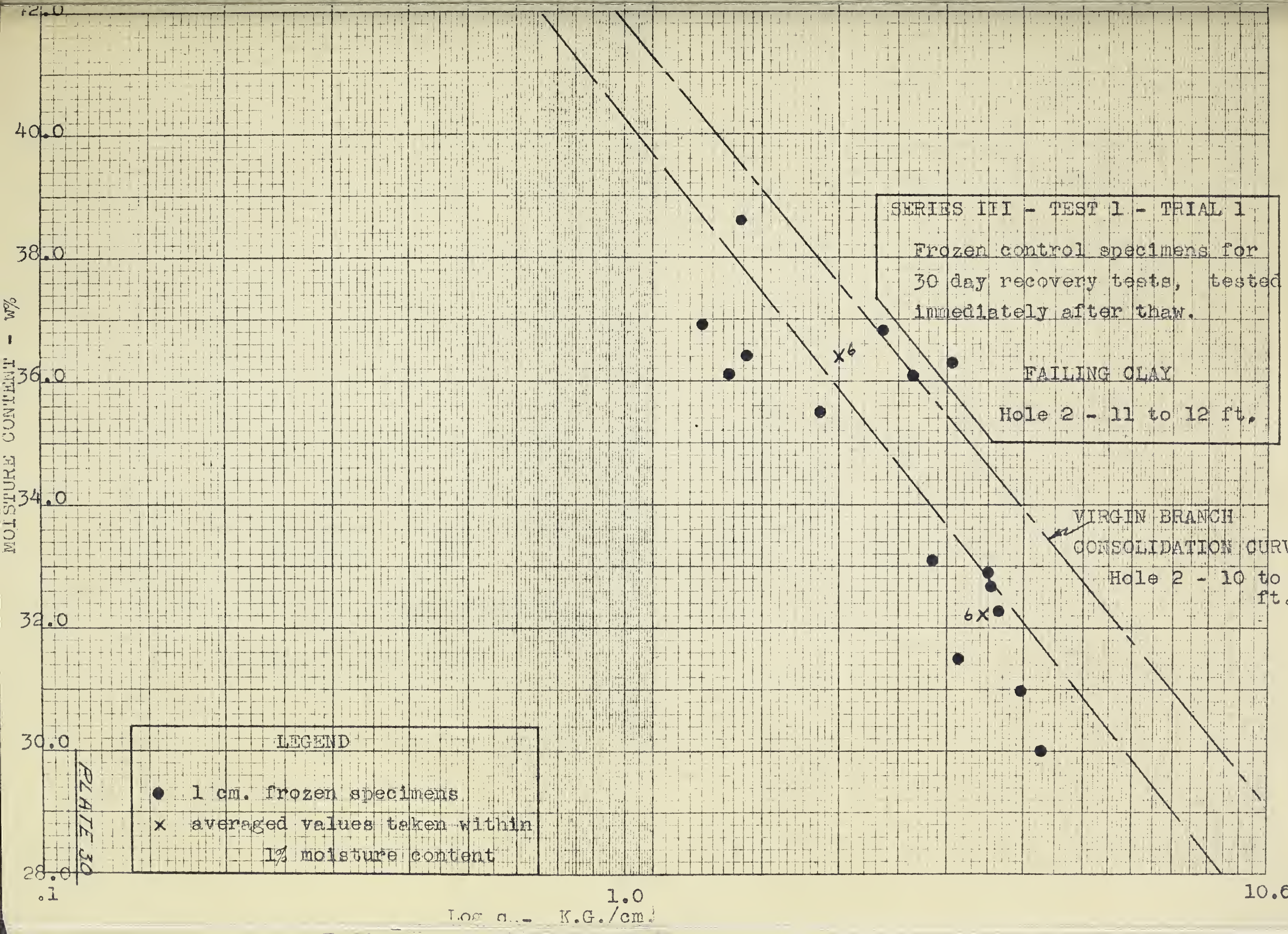

SERIES III - TEST 1 - TRIAL 1
Frozen control specimens for 30 day recovery tests, tested immediately after thaw.
FAILING CLAY
Hole 2 - 11 to 12 ft.
VIRGIN BRANCH CONSOLIDATION CURVE
Hole 2 - 10 to 1 ft.
LEGEND
● 1 cm. frozen specimens
× averaged values taken within 1% moisture content
6
6
MOISTURE CONTENT - W%
40.0
38.0
36.0
34.0
32.0
30.0
28.0
.1
1.0
10.0
Log σ - K.G./cm.

PLATE 30

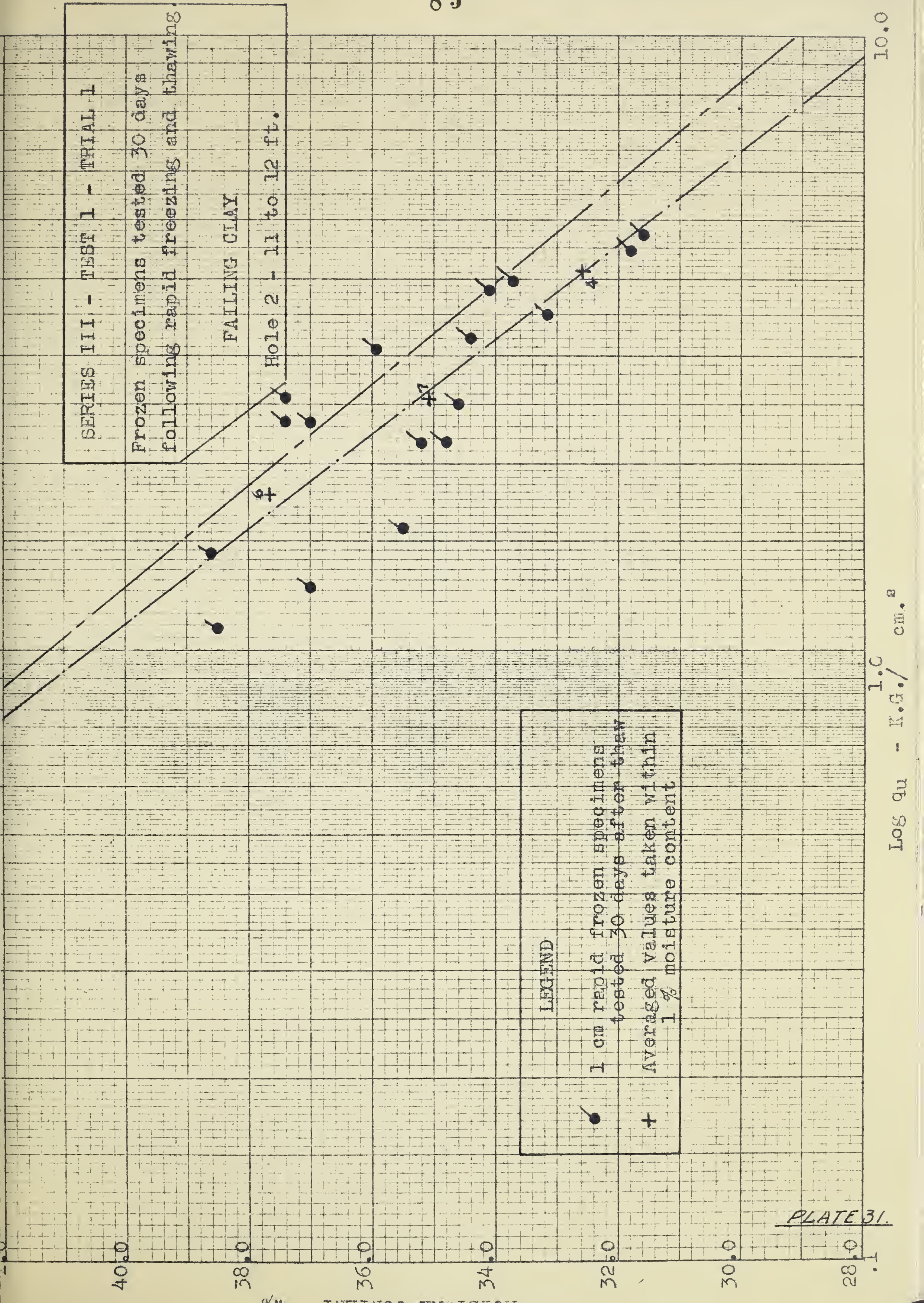

SERIES III - TEST 1 - TRIAL 1
Frozen specimens tested 30 days following rapid freezing and thawing.
FAILING CLAY
Hole 2 - 11 to 12 ft.
LEGEND
1 cm rapid frozen specimens tested 30 days after thaw
+ Averaged values taken within 1 % moisture content
6
7
4
40.0
38.0
36.0
34.0
32.0
30.0
28.0
.1
1.0
10.0
Log qu - K.G./ cm.²

PLATE 31.

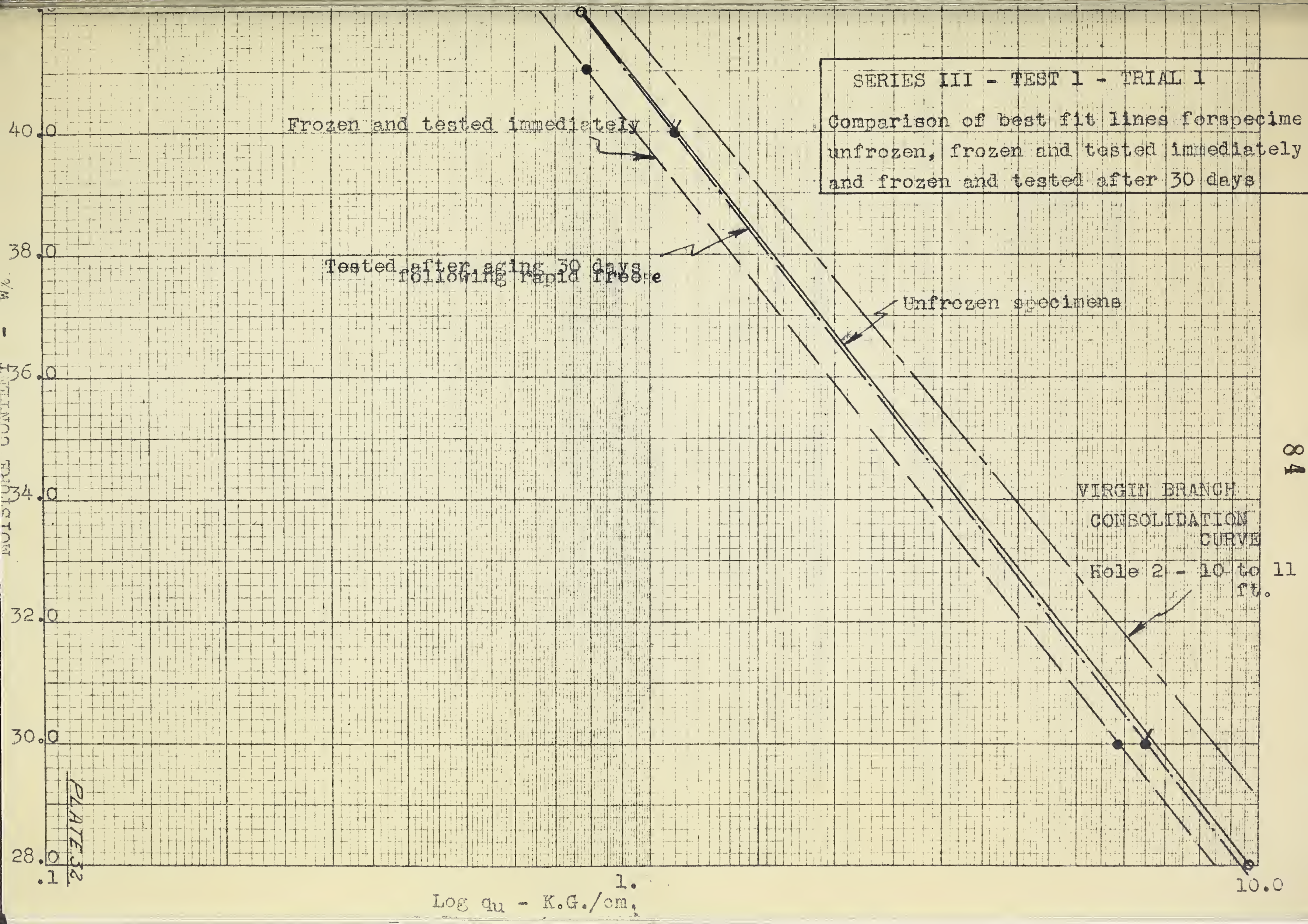

SERIES III - TEST 1 - TRIAL 1
Comparison of best fit lines forspecime unfrozen, frozen and tested immediately and frozen and tested after 30 days
Frozen and tested immediately
Tested after aging 30 days following rapid freeze
Unfrozen specimens
VIRGIN BRANCH CONSOLIDATION CURVE
Hole 2 - 10 to 11 ft.
40.0
38.0
36.0
34.0
32.0
30.0
28.0
.1
1.
10.0
Log qu - K.G./cm.

PLATE 32

SERIES III

Test 1

Trial 2

A second trial was carried out on a sample of Failing Clay to check the recovery of strength 30 days following rapid freezing and thawing, the results of which are given on Plates 33, 34 and 35.

There appeared to be no loss of strength in this case after freezing and thawing, although there was an increase in strength after specimens had aged 30 days as shown by Plate 38. It is therefore suspected that the unfrozen trials were in error. In order for the whole group of specimens to be in error, it is possible that the specimens dried excessively between the time they were tested and the time they were weighed for moisture determinations.

UNIVERSITY OF ALBERTA
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FREEZE - THAW EXPERIMENTS

Site: Failing Drilling Co.
Location: 109 St & Kingsway, Edm.
Hole No. 4 Depth: 7' - 8'
Moist. Cont. 36.7% Temp. 46°F
Date sampled: Nov 5th 54

Test Series: III Test No: 1 Trial No: 2 Date of Test: Jan 13/55

OBJECT:

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.70	69.9	28.4	41.5	.0005	16.6			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a) q_u	Layer (a) W%	Layer (b) q_u	Layer (b) W%	Layer (c) q_u	Layer (c) W%	Layer (c) q_u	Layer (c) W%
UNFROZEN (a) (b) (c) (d)	1 cm.	1	1.05	35.3	2.89	31.5				
		2	1.15	35.3	2.73	31.8				
		3	.80	37.0	.90	36.6				
		4	.95	37.6	.87	37.0				
		5	2.68	31.3	.90	37.8				
		6	3.38	29.2	.61	39.7				
		7	2.35	30.6	2.83	33.8				
		8								
		9								
		10								
		Av.								
	1.4 in.									
FROZEN at -30°C for 24 hrs THAWED at ± 20°C for 24 hrs Tested immediately after thaw	1 cm.	1	.92	36.0	2.29	31.7				
		2	.65	36.2	2.49	32.0				
		3	.71	37.8	2.03	33.9				
		4	.72	37.8	2.23	33.9				
		5	2.41	31.7	.65	38.1				
		6	1.88	33.1	.80	37.1				
		7			1.02	36.7				
		8			1.35	34.5				
		9								
		10								
		Av.								
	1.4 in.									

REMARKS

1. This is control data for 30 day recovery tests given on next page, and for 60 and 90 day tests which follow.
2. Unfrozen specimen No. 7, layer (b) and frozen specimen No. 8, layer (b) appear to be in error and should be discarded.

UNIVERSITY OF ALBERTA
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FREEZE - THAW EXPERIMENTS

Site: Failing Drilling Co.
Location: 109 St & Kingsway, Edm.
Hole No. 4 Depth: 7'-8'
Moist. Cont. 36.7% Temp. 46°F
Date sampled: Nov 5th 54

Test Series: III Test No: 1 Trial No: 2 Cont'd Date of Test: Feb 14/55

OBJECT:
As for Trial 1.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.70	69.9	28.4	41.5	.0005	16.6			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (c)	
			q_u	w%	q_u	w%	q_u	w%	q_u	w%
		1								
UNFROZEN		2								
		3								
		4								
(a)	1 cm.	5								
(b)		6								
		7								
(c)		8								
(d)		9								
		10								
		Av.								
	1.4 in.									
		1	1.12	37.9	1.09	35.4	2.59	30.7		
FROZEN		2	.90	37.5	1.27	35.2	2.40	32.1		
		3	1.47	35.8	1.06	35.6	2.25	32.6		
at -30 °C		4	1.15	35.5	1.03	36.0	2.20	32.8		
for 24 hrs	1 cm.	5	2.02	32.45			1.72	35.4		
		6					1.67	36.3	(discarded)	
THAWED		7								
		8								
at +20 °C		9								
		10								
for 24 hrs		Av.								
Tested 30 days after thaw	1.4 in.									

REMARKS

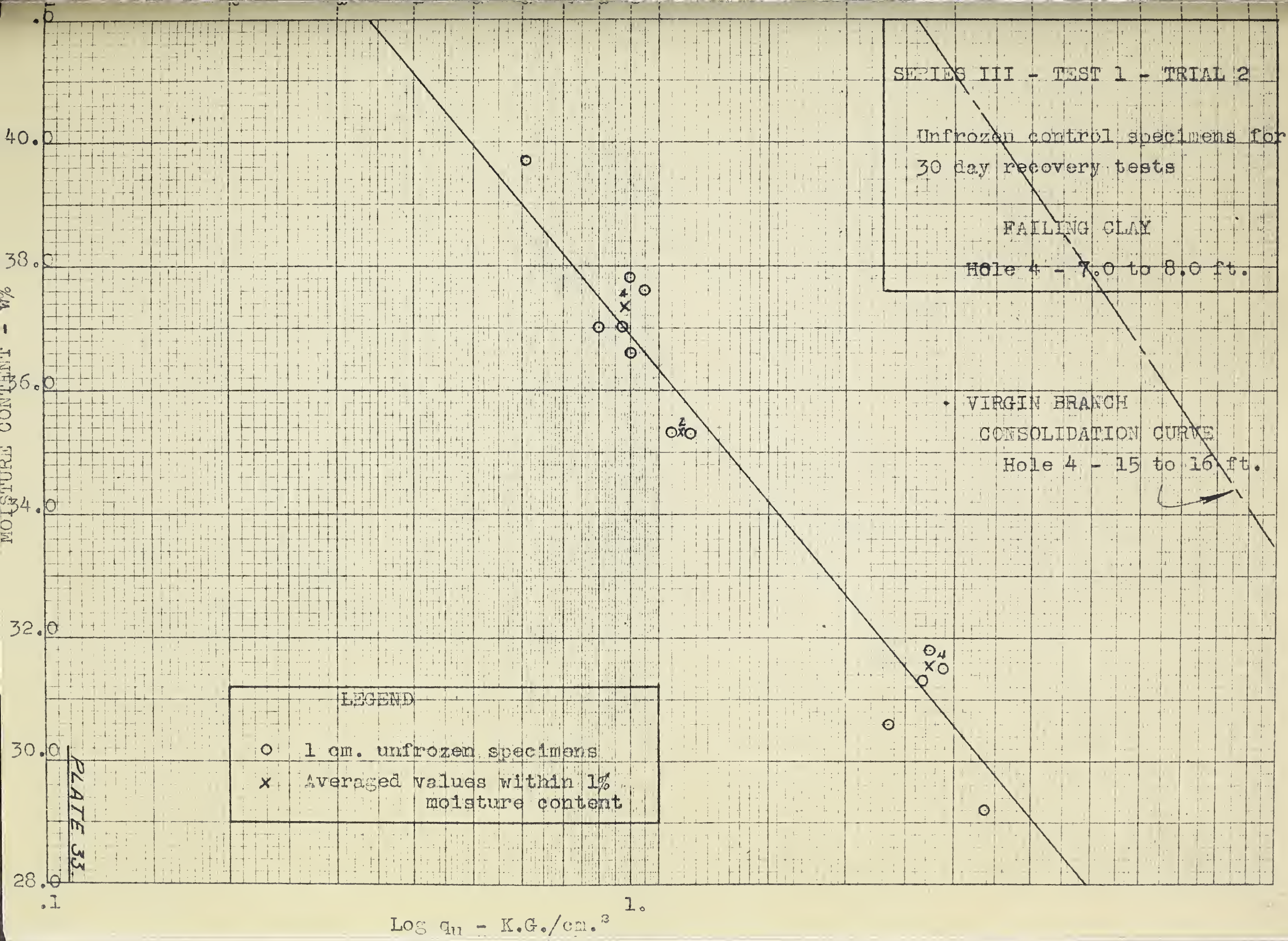
SERIES III - TEST 1 - TRIAL 2
Unfrozen control specimens for
30 day recovery tests
FAILING CLAY
Hole 4 - 7.0 to 8.0 ft.
VIRGIN BRANCH
CONSOLIDATION CURVE
Hole 4 - 15 to 16 ft.
LEGEND
○ 1 cm. unfrozen specimens
× Averaged values within 1% moisture content
MOISTURE CONTENT - W%
40.0
38.0
36.0
34.0
32.0
30.0
28.0
.1
1.
Log q_u - K.G./cm.²
4
2
4

PLATE 33.

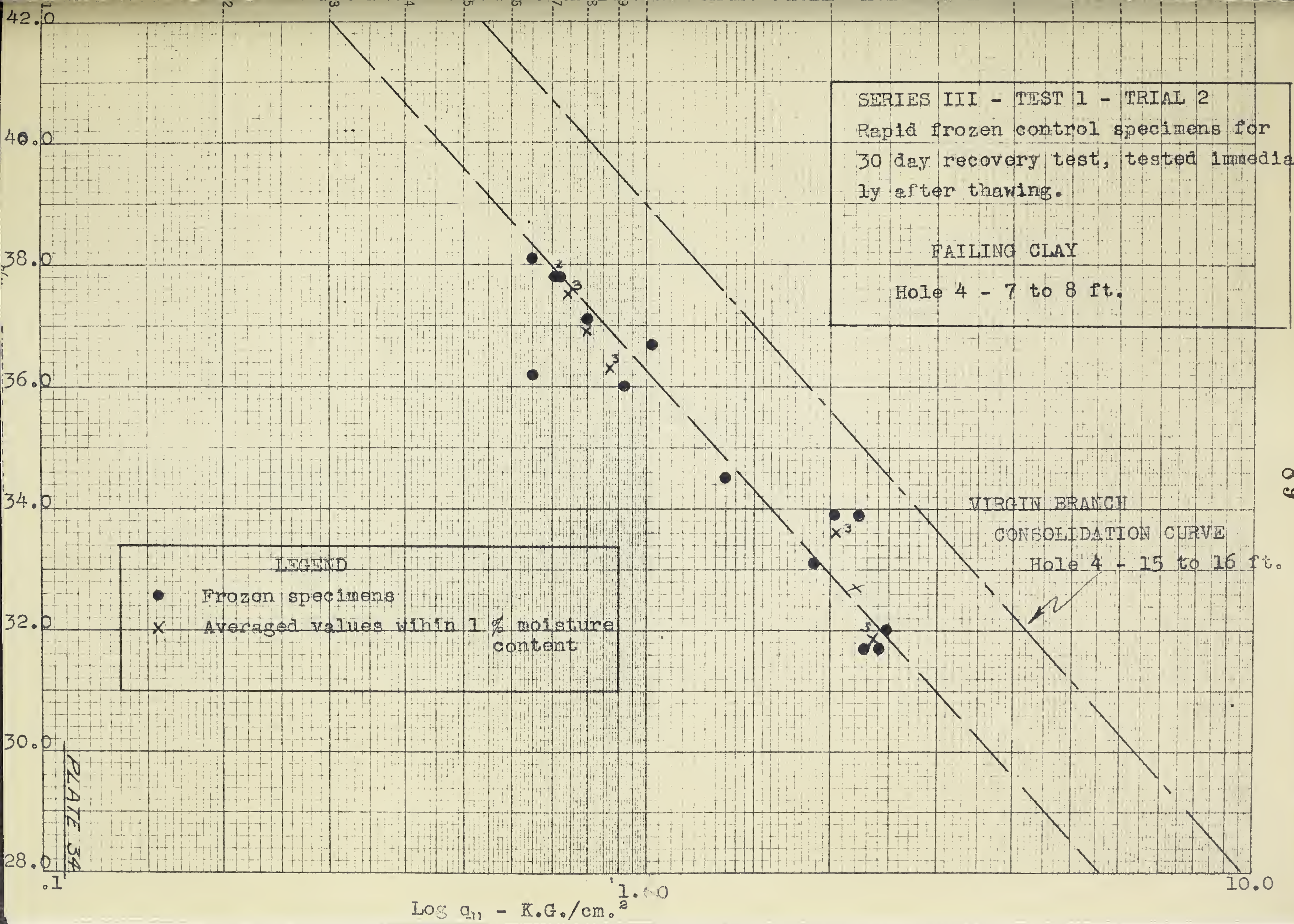
SERIES III - TEST 1 - TRIAL 2
Rapid frozen control specimens for 30 day recovery test, tested immediat ly after thawing.
FAILING CLAY
Hole 4 - 7 to 8 ft.
VIRGIN BRANCH CONSOLIDATION CURVE
Hole 4 - 15 to 16 ft.
LEGEND
● Frozen specimens
X Averaged values within 1 % moisture content
3
42.0
40.0
38.0
36.0
34.0
32.0
30.0
28.0
.1
1.0
10.0
2
3
4
5
6
7
8
9
Log q_u - K.G./cm.²

PLATE 34

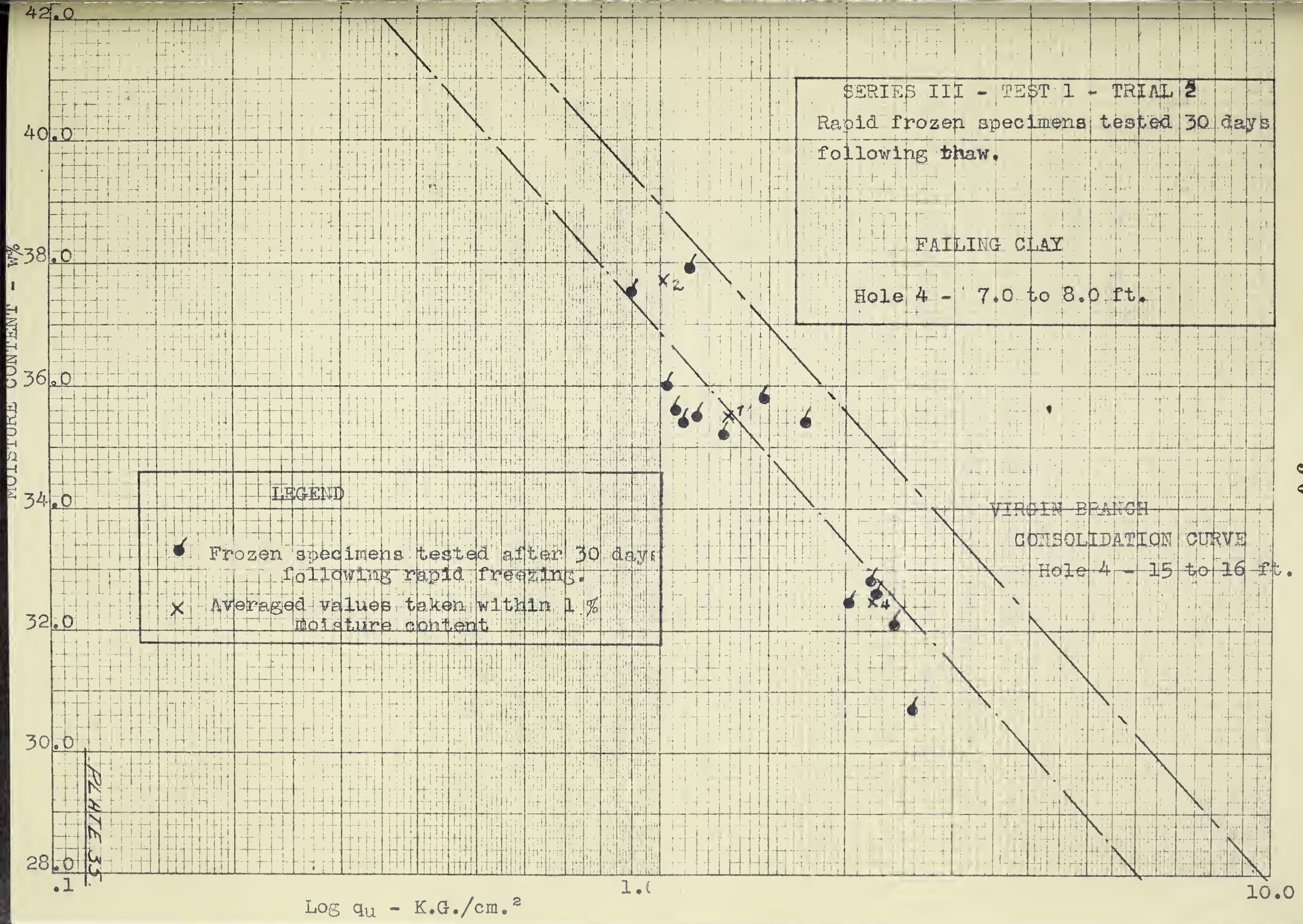
SERIES III - TEST 1 - TRIAL 2
Rapid frozen specimens tested 30 days following thaw.
FAILING CLAY
Hole 4 - 7.0 to 8.0 ft.
VIRGIN BRANCH CONSOLIDATION CURVE
Hole 4 - 15 to 16 ft.
LEGEND
Frozen specimens tested after 30 days following rapid freezing.
Averaged values taken within 1 % moisture content
×2
×1
×4
MOISTURE CONTENT - W%
42.0
40.0
38.0
36.0
34.0
32.0
30.0
28.0
.1
1.0
10.0
Log qu - K.G./cm.²

PLATE 35.

Test 2 and 3

Trial 1

Some specimens from the sample used for Test 1, Trial 2, above were allowed to age for a longer period, some were tested after 60 days while the remainder were tested after 90 days. The results are given on Plates 36 and 37. The best fit lines have been transposed to Plate 38, so that they may be compared with the 30 day recovery tests.

It can only be concluded from this trial that there is a recovery of strength with time following rapid freezing and thawing, but the amount of recovery expressed as a percentage of initial strength cannot be determined since the initial strength in this case was ruled to be in error.

Neither can any additional recovery be detected beyond the first 30 days. The strength curves for the 30, 60 and 90 day recovery tests appear to be nearly coincident.

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FREEZE - THAW EXPERIMENTS

Site: Failing Drilling Co.
Location: 109 St & Kingsway, Edm.
Hole No. 4 Depth: 7' - 8'
Moist. Cont. 36.7% Temp. 46°F
Date sampled: Nov 5th 54

Test Series: III Test No: 2 Trial No: 1 Date of Test: Mar 18/55

OBJECT: To determine whether there is a recovery of strength of a clay 60 days following one cycle of rapid freezing and thawing.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.70	69.9	28.4	41.5	.0005	16.6			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.^{2}

Test conditions	Size of Specimen		Layer (a) q_u	Layer (a) W%	Layer (b) q_u	Layer (b) W%	Layer (c) q_u	Layer (c) W%	Layer (c) q_u	Layer (c) W%
UNFROZEN (a) (b) (c) (d)	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									
FROZEN at -30 °C for 24 hrs THAWED at ±20 °C for 24 hrs Tested 60 days after thaw	1 cm.	1	1.66	34.8	1.74	32.6	1.22	30.9	(discarded)	
		2	1.49	35.9	2.62	32.45	1.43	81.0		
		3	0.94	35.6	2.32	31.0	1.20	35.6		
		4	2.83	32.4	1.88	32.2	1.19	35.5		
		5	.92	36.6	1.00	37.4				
		6	2.13	33.75						
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS

1. Series III, Test 1, Trial 2 is used as control for this test.
2. Sample appeared to have nugget structure. Soft consistency, W% near W_p. Sample compressed 7.4 mm. in a length of 1½ ins. when cut into specimen tubes.
3. Specimen 3, layer (a) split up the middle upon failing; Specimen 4 showed signs of end restraint.
4. Specimen 2, layer (b) showed signs of end restraint.
5. Specimen 2, layer (c) appears to be grossly in error.

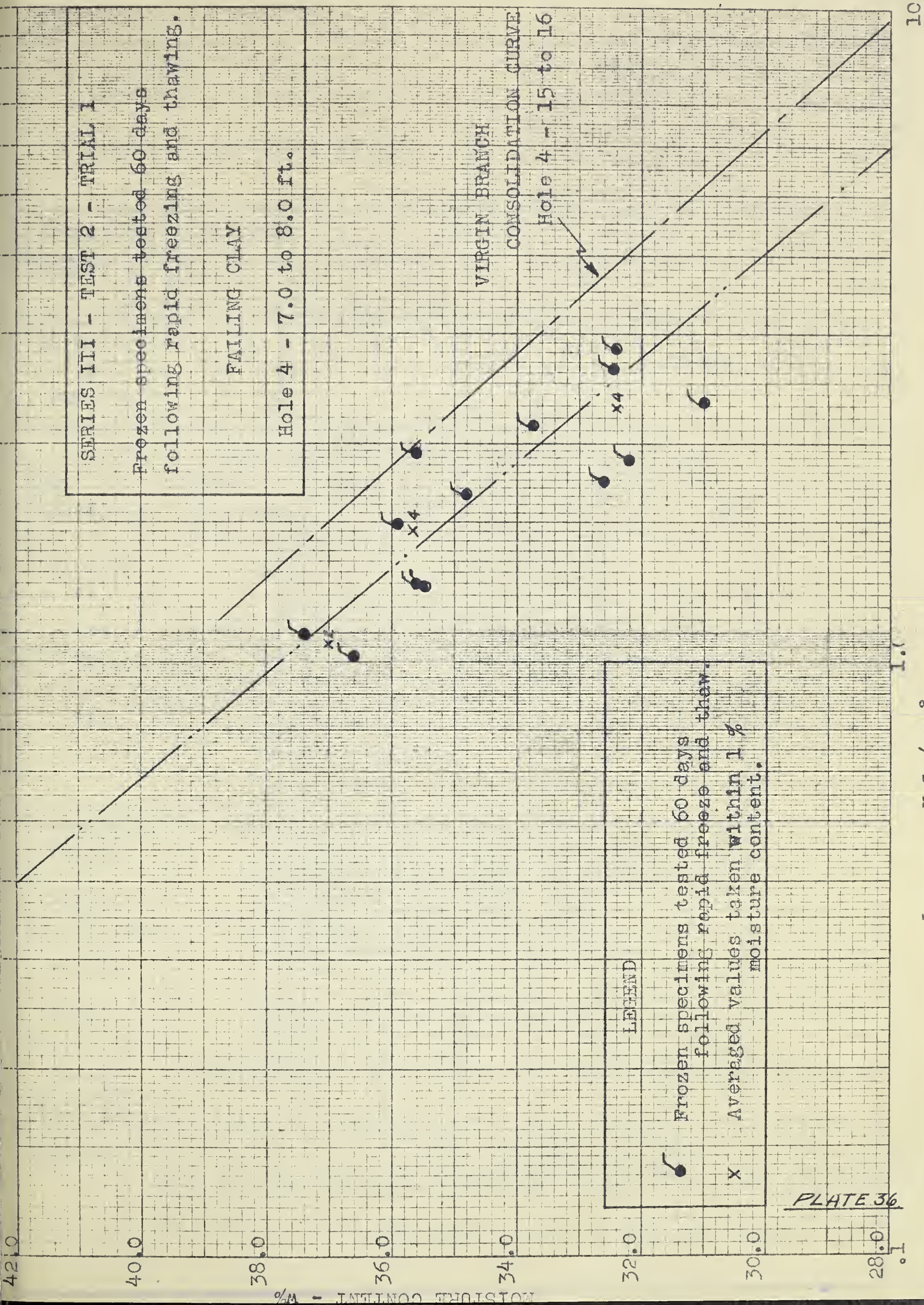
SERIES III - TEST 2 - TRIAL 1
~~Frozen specimens tested 60 days~~
following rapid freezing and thawing.
FAILING CLAY
Hole 4 - 7.0 to 8.0 ft.
VIRGIN BRANCH
CONSOLIDATION CURVE
Hole 4 - 15 to 16 ft.
LEGEND
Frozen specimens tested 60 days following rapid freeze and thaw.
X Averaged values taken within 1 % moisture content.
x2
x4
x4
Log qu - K.G./cm.²
.1
1.0
10.0
MOISTURE CONTENT - W%
42.0
40.0
38.0
36.0
34.0
32.0
30.0
28.0

PLATE 36.

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FREEZE - THAW EXPERIMENTS

Site: Failing Drilling Co.
Location: 109 St & Kingsway, Edm.
Hole No. 4 Depth: 7' - 8'
Moist. Cont. 36.7% Temp. 46F
Date sampled: Nov 5th 54

Test Series: III Test No: 3 Trial No: 1 Date of Test: Apr 23/55

OBJECT: To determine whether there is a recovery of strength of a clay 90 days following one cycle of rapid freezing and thawing.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.70	69.9	28.4	41.5	.0005	16.6			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.^{2}

Test conditions	Size of Specimen		Layer (a) q_u	Layer (a) W%	Layer (b) q_u	Layer (b) W%	Layer (c) q_u	Layer (c) W%	Layer (c) q_u	Layer (c) W%
UNFROZEN (a) (b) (c) (d)	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									
FROZEN at -30 °C for 24 hrs THAWED at ±20 °C for 24 hrs Tested 90 days after thaw	1 cm.	1	2.14	32.1	2.61	31.5	1.14	35.7		
		2	2.10	33.1	1.32	35.9	1.15	35.5		
		3	2.30	33.5	2.05	34.7	1.17	36.2		
		4	1.15	37.2	1.84	35.1				
		5	1.10	37.5	1.33	36.4				
		6	2.58	31.5						
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS
1. Series III, Test 1, trial 2 is used as control for this test.
2. Specimen 1, layer (b) was difficult to remove from the specimen tube and may therefore have been compressed in the process.
3. Specimen 1, layer (a) showed signs of end restraint.

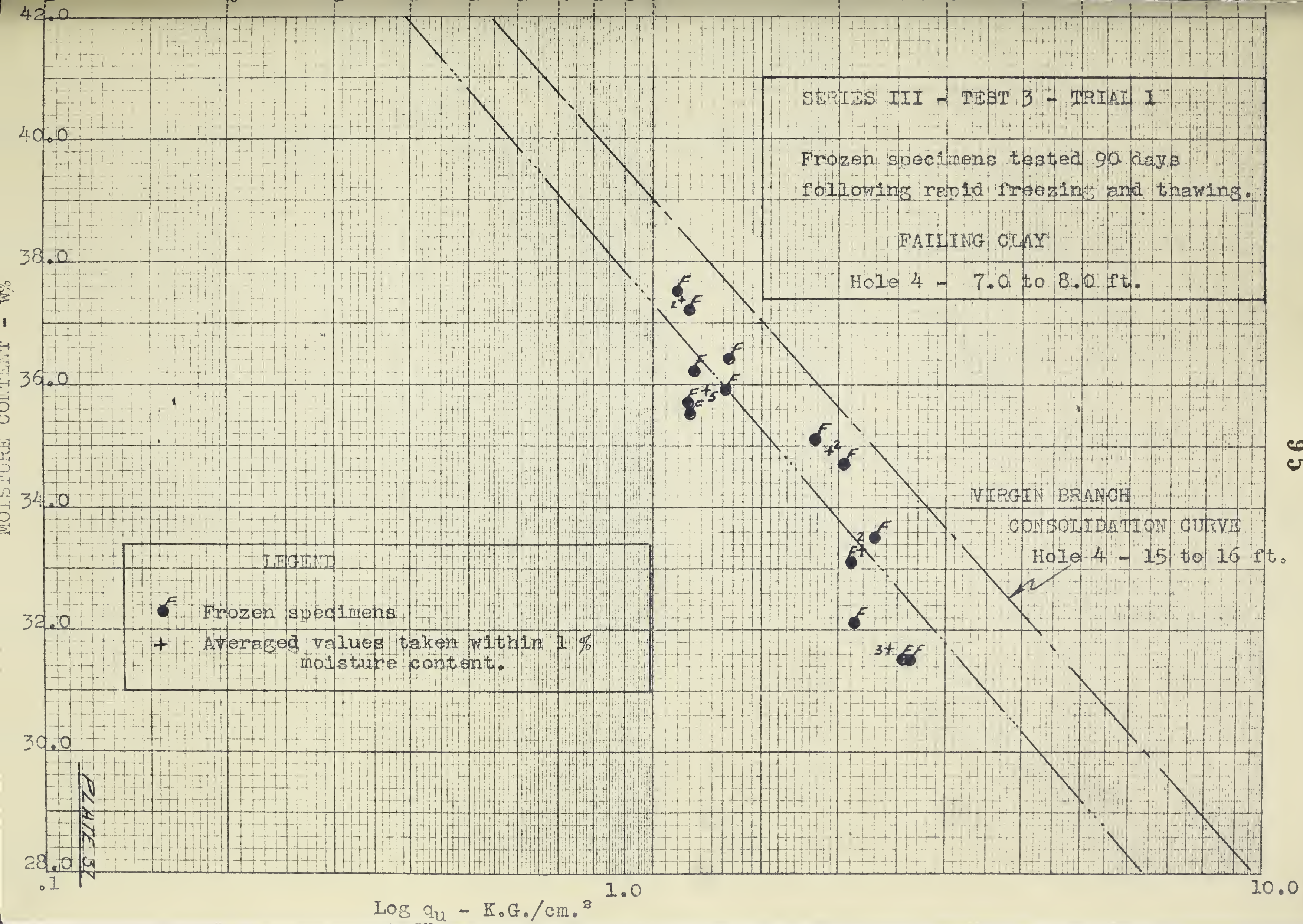
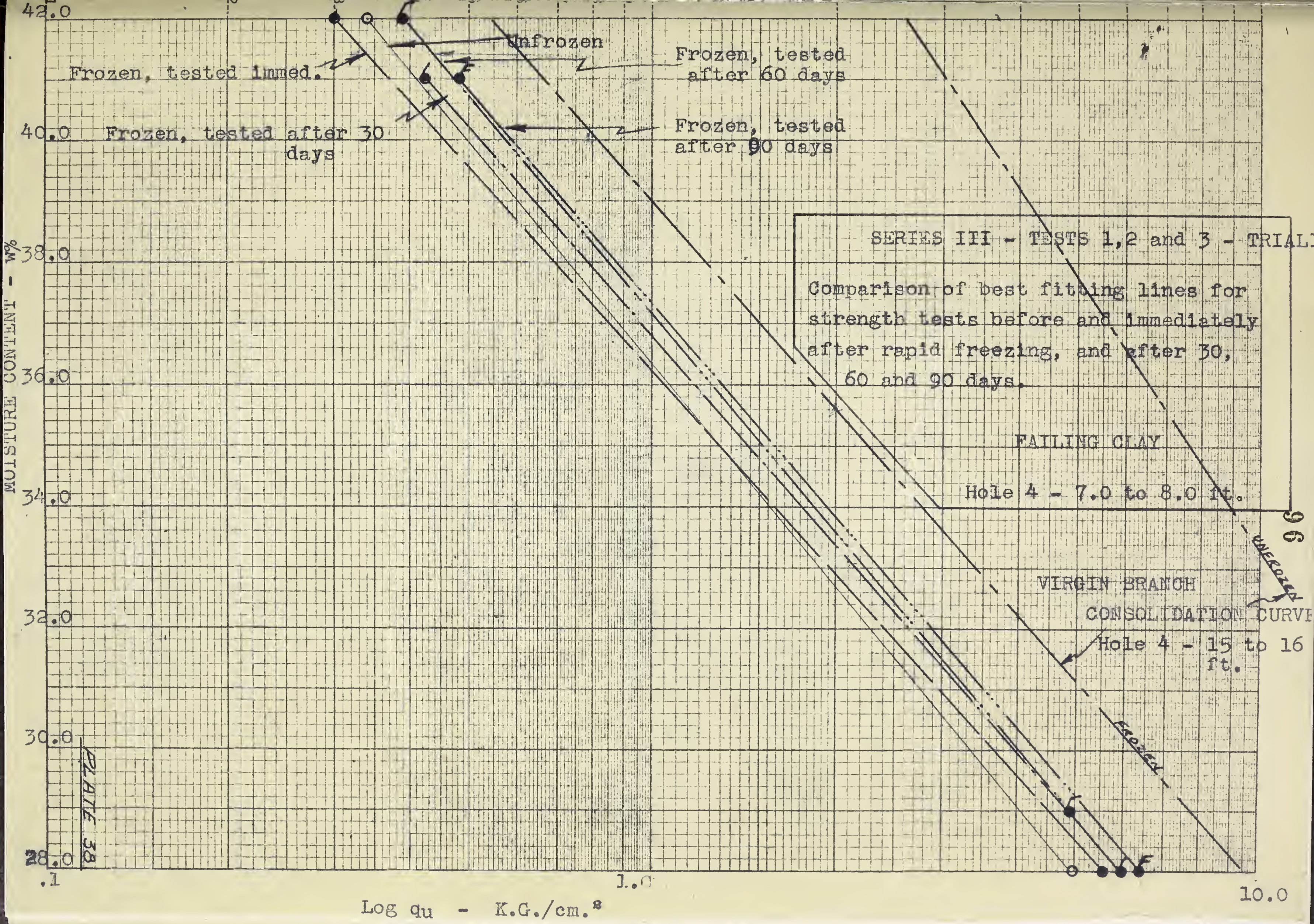

SERIES III - TEST 3 - TRIAL 1
Frozen specimens tested 90 days following rapid freezing and thawing.
FAILING CLAY
Hole 4 - 7.0 to 8.0 ft.
VIRGIN BRANCH
CONSOLIDATION CURVE
Hole 4 - 15 to 16 ft.
LEGEND
F Frozen specimens
+ Averaged values taken within 1 % moisture content.
42.0
40.0
38.0
36.0
34.0
32.0
30.0
28.0
.1
1.0
10.0
Log q_u - K.G./cm.²
MOISTURE CONTENT - W%
F
2
3
5

PLATE 37

PLATE 38



SERIES III - TESTS 1,2 and 3 - TRIAL

Comparison of best fitting lines for strength tests before and immediately after rapid freezing, and after 30, 60 and 90 days.

FAILING CLAY

Hole 4 - 7.0 to 8.0 ft.

SERIES III

Test 2 and 3

Trial 2

A second trial was carried out on Failing clay to determine the recovery of strength after 60 and 90 days following rapid freezing and thawing, the results of which are given on Plates 39 to 43 inclusive.

This sample exhibited an initial loss of strength of approximately 50°/o, of which loss only 10°/o was recovered after 60 days and 20°/o after 90 days.

Although there is insufficient data for conclusive proof, it appears that some clay continues to recover strength with time at least beyond the first 60 days.

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FREEZE - THAW EXPERIMENTS

Site: Failing Drilling Co.
Location: 109 St & Kingsway, Edm.
Hole No. 3 Depth: 12'-13'
Moist. Cont. 43.3% Temp. 46°F
Date sampled: Oct 30th 54

Test Series: III Test No: 2 Trial No: 2 Date of Test: Jan 17/55

OBJECT:
As for trial 1.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.71	63.5	28.4	35.1			.37	3.24×10^{-8}	CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.^{2}

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (c)	
			q_u	W%	q_u	W%	q_u	W%	q_u	W%
UNFROZEN (a) (b) (c) (d)	1 cm.	1	2.03	37.5	2.38	41.4	2.49	38.2		
		2	1.65	39.4	2.43	39.5	2.53	36.7		
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									
FROZEN at -30 °C for 24 hrs THAWED at +20 °C for 24 hrs Tested immediately after thaw	1 cm.	1	2.48	35.5	2.05	36.4				
		2	2.23	34.0						
		3	2.23	36.5						
		4	2.02	35.0						
		5	2.13	35.2						
		6	2.40	34.7						
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS
1. This control data for 60 day recovery tests given on next page and for 90 day recovery tests which follow.
2. Frozen specimen No. 1, layer (a) showed signs of end restraint.

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FREEZE - THAW EXPERIMENTS

Site: Failing Drilling Co.
Location: 109 St & Kingsway, Edm.
Hole No. 3 Depth: 12'-13'
Moist. Cont. 43.3% Temp. 46 °F
Date sampled: Oct 30th 54

Test Series: III Test No: 2 Trial No: 2 Cont'd Date of Test: 5

OBJECT: As for Trial 1

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.71	63.5	28.4	35.1			.37	3.24×10^{-8}	CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.^{2}

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (c)	
			q_u	W%	q_u	W%	q_u	W%	q_u	W%
UNFROZEN (a) (b) (c) (d)	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									
FROZEN at -30 °C for 24 hrs THAWED at ±20 °C for 24 hrs Tested 60 days after thaw	1 cm.	1	1.98	35.6						
		2	1.97	36.0						
		3	1.98	36.2						
		4	2.10	36.4						
		5	1.98	36.7						
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS

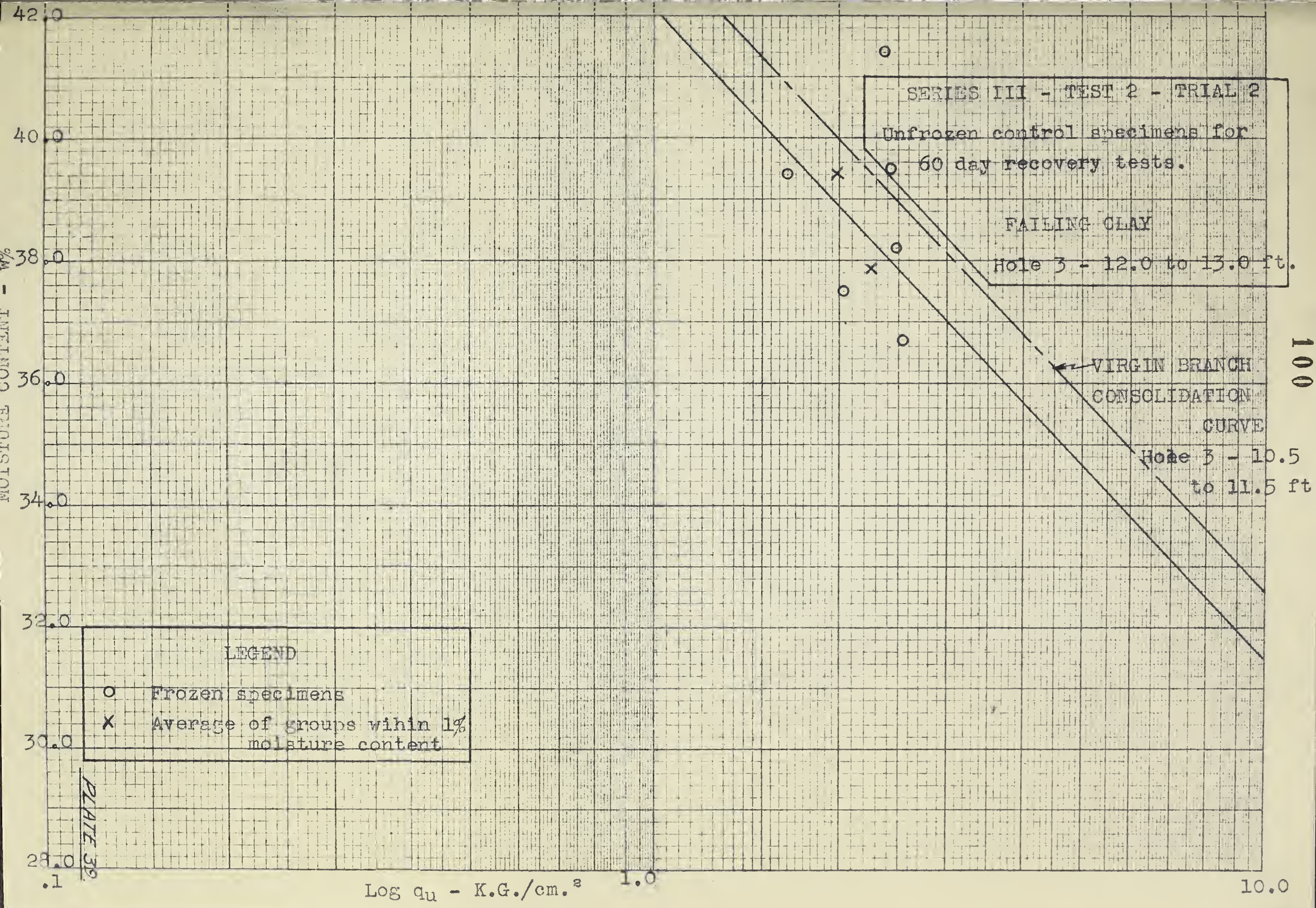
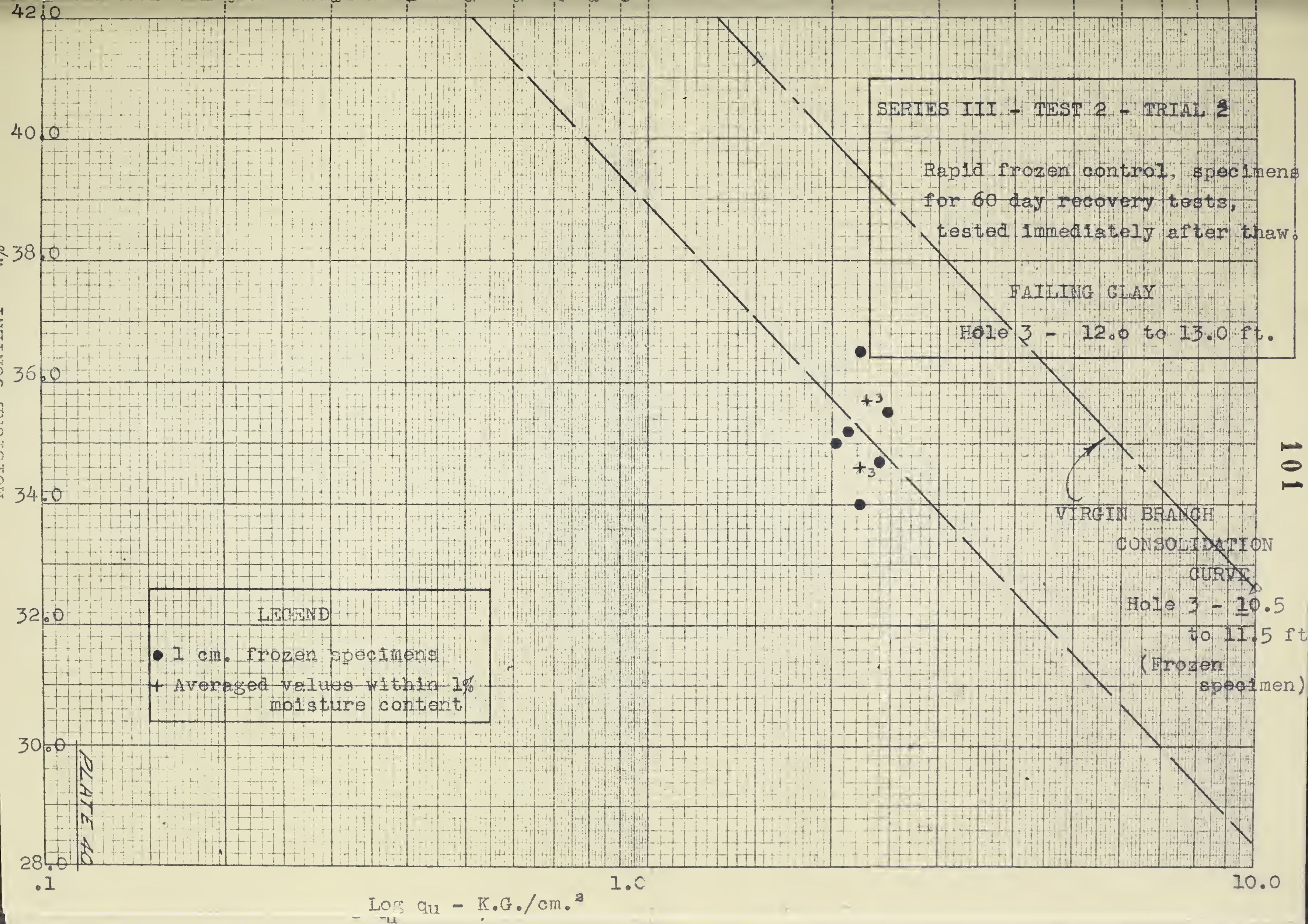

SERIES III - TEST 2 - TRIAL 2
Unfrozen control specimens for
60 day recovery tests.
FAILING CLAY
Hole 3 - 12.0 to 13.0 ft.
VIRGIN BRANCH
CONSOLIDATION
CURVE
Hole 3 - 10.5
to 11.5 ft.
LEGEND
o Frozen specimens
X Average of groups wihin 1% moisture content
42.0
40.0
38.0
36.0
34.0
32.0
30.0
28.0
%W - MOISTURE CONTENT
.1
1.0
10.0
Log q_u - K.G./cm.²

PLATE 39



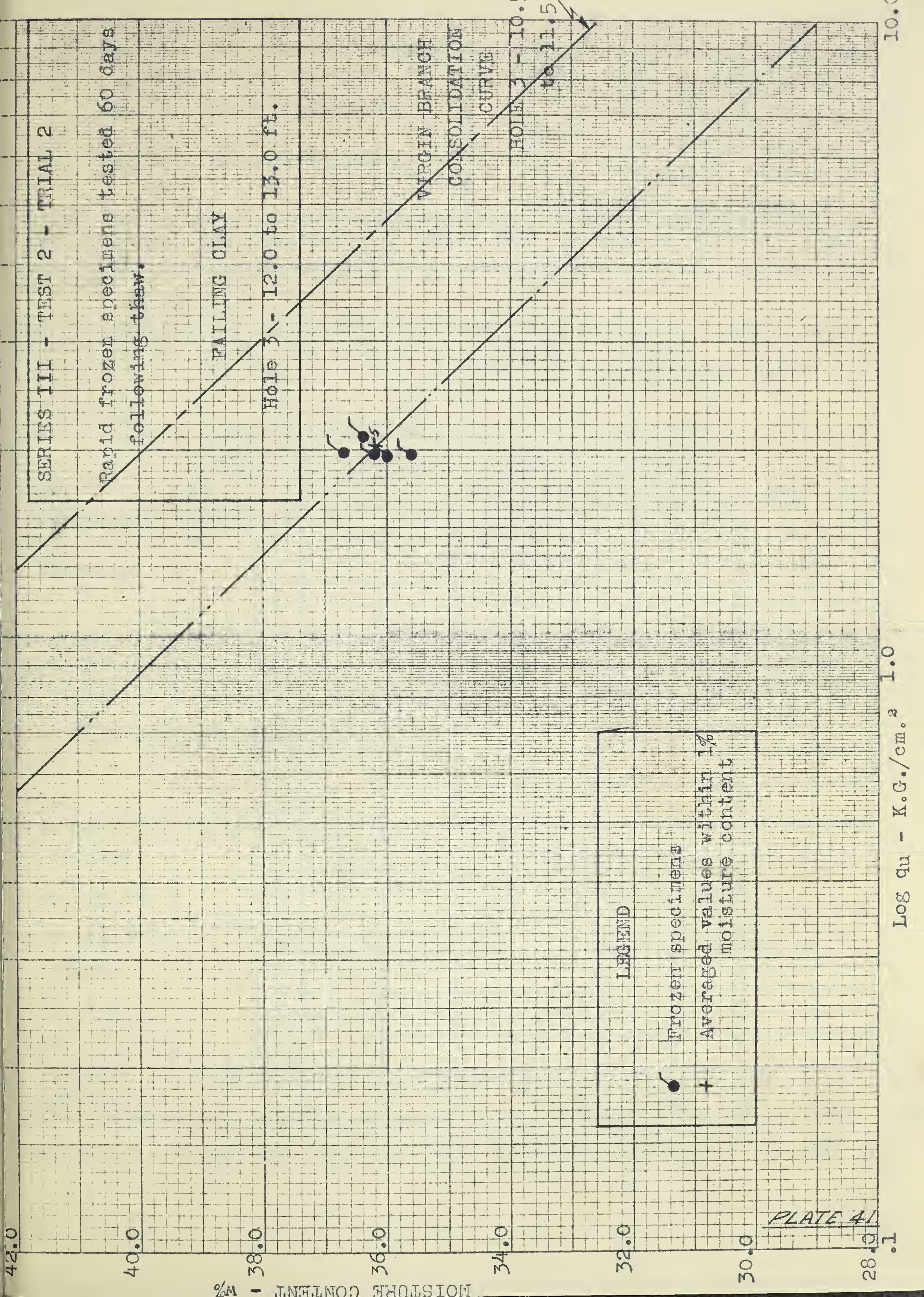
SERIES III - TEST 2 - TRIAL 2

Rapid frozen control, specimens for 60 day recovery tests, tested immediately after thaw.

FAILING CLAY

Hole 3 - 12.0 to 13.0 ft.

PLATE 40


SERIES III - TEST 2 - TRIAL 2
Rapid frozen specimens tested 60 days following thaw.
FAILING CLAY
Hole 3 - 12.0 to 13.0 ft.
VIRGIN BRANCH CONSOLIDATION CURVE
HOLE 3 - 10.5 to 11.5 ft
LEGEND
Frozen specimens
Averaged values within 1% moisture content
MOISTURE CONTENT - W%
42.0
40.0
38.0
36.0
34.0
32.0
30.0
28.0
Log qu - K.G./cm.²
.1
1.0
10.0
PLATE 41

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FREEZE - THAW EXPERIMENTS

Site: Failing Drilling Co.
Location: 109 St & Kingsway, Edm.
Hole No. 3 Depth: 12'-13ft.
Moist. Cont. 43.3% Temp. 46°F
Date sampled: Oct 30th 54

Test Series: III Test No: 3 Trial No: 2 Date of Test: Apr 23/55

OBJECT:
As for Trial 1.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.74	63.5	28.4	35.1			.37	3.24×10^{-8}	CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a) q_u	Layer (a) w%	Layer (b) q_u	Layer (b) w%	Layer (c) q_u	Layer (c) w%	Layer (c) q_u	Layer (c) w%
UNFROZEN (a) (b) (c) (d)	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									
FROZEN at -30 °C for 24 hrs THAWED at ±20 °C for 24 hrs Tested 90 days after thaw	1 cm.	1	1.43	38.1	1.96	35.9				
		2	2.39	34.7	2.23	35.7				
		3	1.53	39.9	1.85	37.1				
		4			1.23	39.6				
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS 1. Series III, Test 2, trial 2 is used as control for this test.

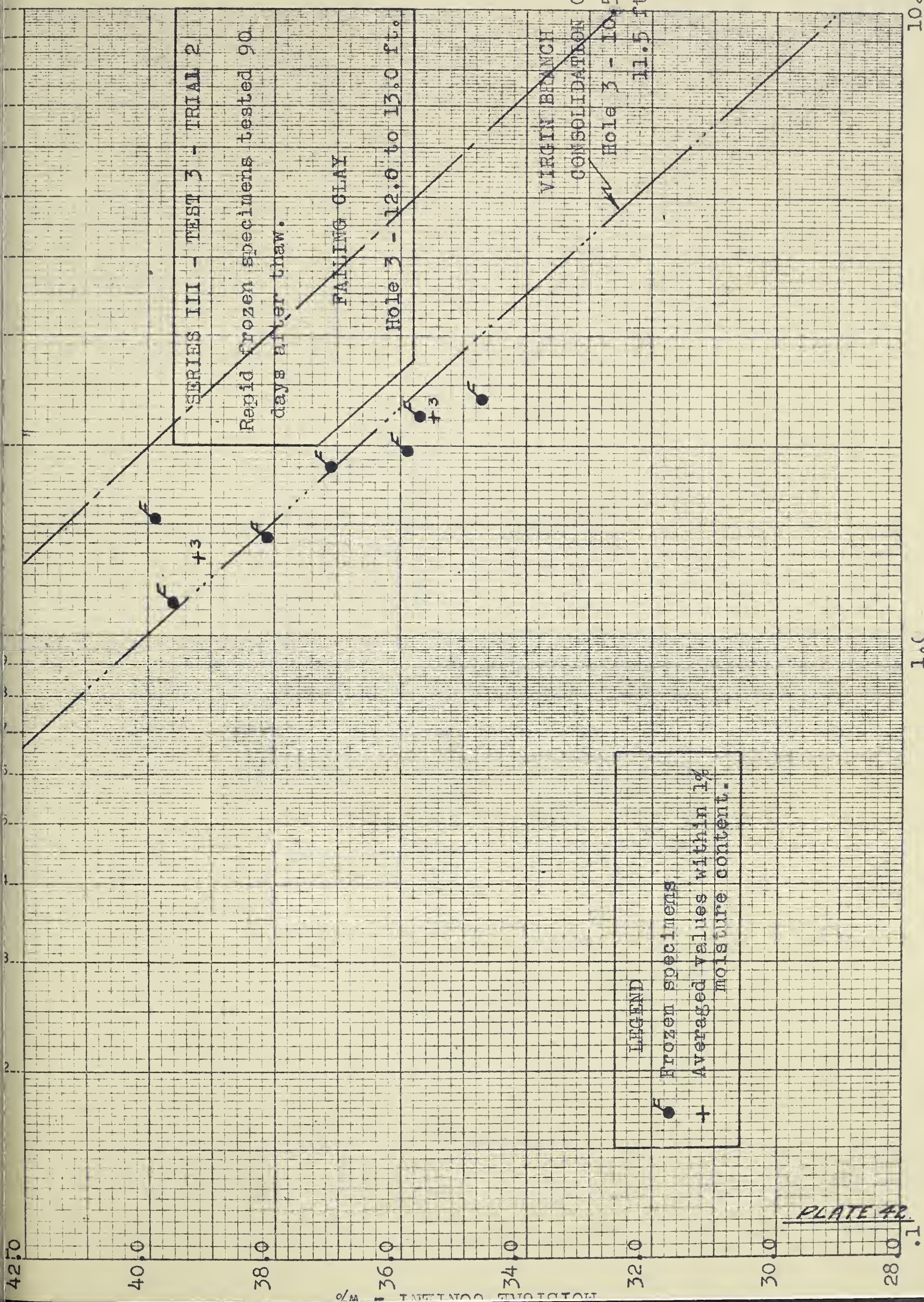
SERIES III - TEST 3 - TRIAL 2
Rapid frozen specimens tested 90 days after thaw.
FALLING CLAY
Hole 3 - 12.0 to 13.0 ft.
VIRGIN BRANCH CONSOLIDATION CURV
Hole 3 - 10.5 to 11.5 ft.
LEGEND
Frozen specimens
Averaged values within 1% moisture content.
F
+3
Log q_u - K.G./cm.^{2}
.1
1.0
10.0
42.0
40.0
38.0
36.0
34.0
32.0
30.0
28.0

PLATE 42

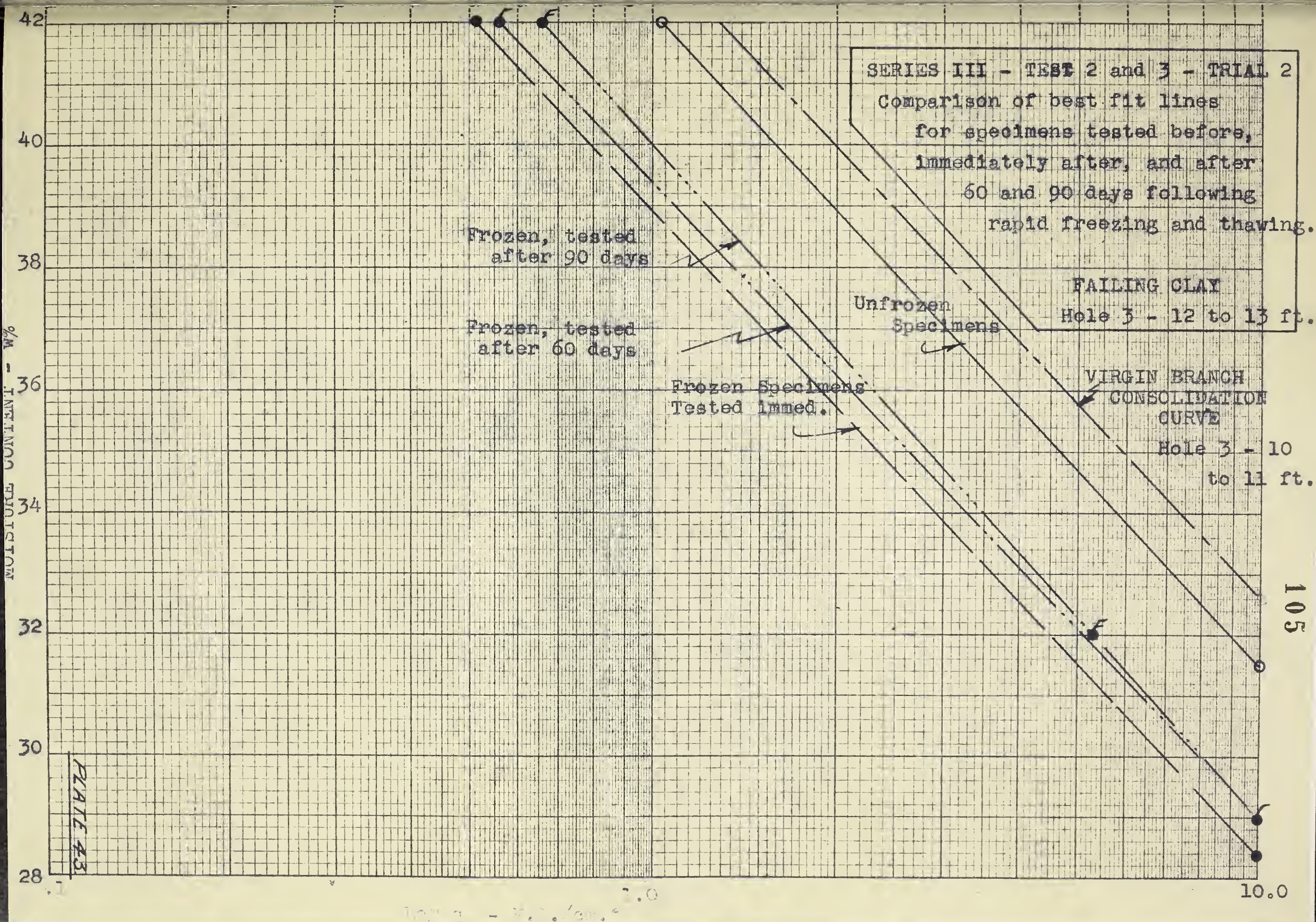
SERIES III - TEST 2 and 3 - TRIAL 2
Comparison of best fit lines
for specimens tested before,
immediately after, and after
60 and 90 days following
rapid freezing and thawing.
FAILING CLAY
Hole 3 - 12 to 13 ft.
Frozen, tested
after 90 days
Frozen, tested
after 60 days
Unfrozen
Specimens
Frozen Specimens
Tested Immed.
VIRGIN BRANCH
CONSOLIDATION
CURVE
Hole 3 - 10
to 11 ft.
42
40
38
36
34
32
30
28
MOISTURE CONTENT - W%
.1
1.0
10.0

PLATE 43

SERIES IV - To determine whether there is a recovery of strength with time following slow freezing and thawing.

The sample of Failing clay used for Series I, Test 3, had been divided into five groups of specimens, three of which had been set aside in sealed containers to age for 30, 60 and 90 days after being frozen and thawed slowly.

The results of these tests are given on Plates 44, 45 and 46. The best fit lines for Series I, Test 3 are used for control and are presented on Plate 47, along with the best fit lines for this test for comparison.

There appears to be a loss of strength of approximately 35°/o as a result of slow freezing and thawing. Approximately 20°/o of this loss was recovered after 30 days. Beyond that time there appears to be no additional recovery of strength.

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FREEZE - THAW EXPERIMENTS

Site: Failing Drilling Co.
Location: 109 St & Kingsway, Edm.
Hole No. 4 Depth: 8 ft- 9 ft.
Moist. Cont. 36.3% Temp. 46°F
Date sampled: Nov 5th 54

Test Series: IV Test No: 1 Trial No: 1 Date of Test: Feb 23/55

OBJECT: To determine whether there is a recovery of strength of clay with time following one cycle of slow freezing and thawing.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.70	69.9	28.4	41.5	.0005	16.6			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (c)	
			q_u	w%	q_u	w%	q_u	w%	q_u	w%
UNFROZEN (a) (b) (c) (d)	1 cm.	1								
		2								
		3								
		4								
		5								
		6								
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									
FROZEN at ±25 °C for 24 hrs THAWED at ±35 °C for 24 hrs Tested 30 days after thaw	1 cm.	1	.75	38.65	1.47	32.9	1.40	35.5		
		2	.72	40.50	1.00	38.17	1.42	35.5		
		3	1.30	35.5	.88	39.2	1.90	29.5		
		4	1.53	37.3	.70	38.8	1.43	37.2		
		5	1.02	37.85	.68	37.2				
		6	1.03	34.25						
		7								
		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS
1. Series I, Test 3 is used for control for this test.
2. Specimen No. 3, layer (c) appears too low and should be discarded.

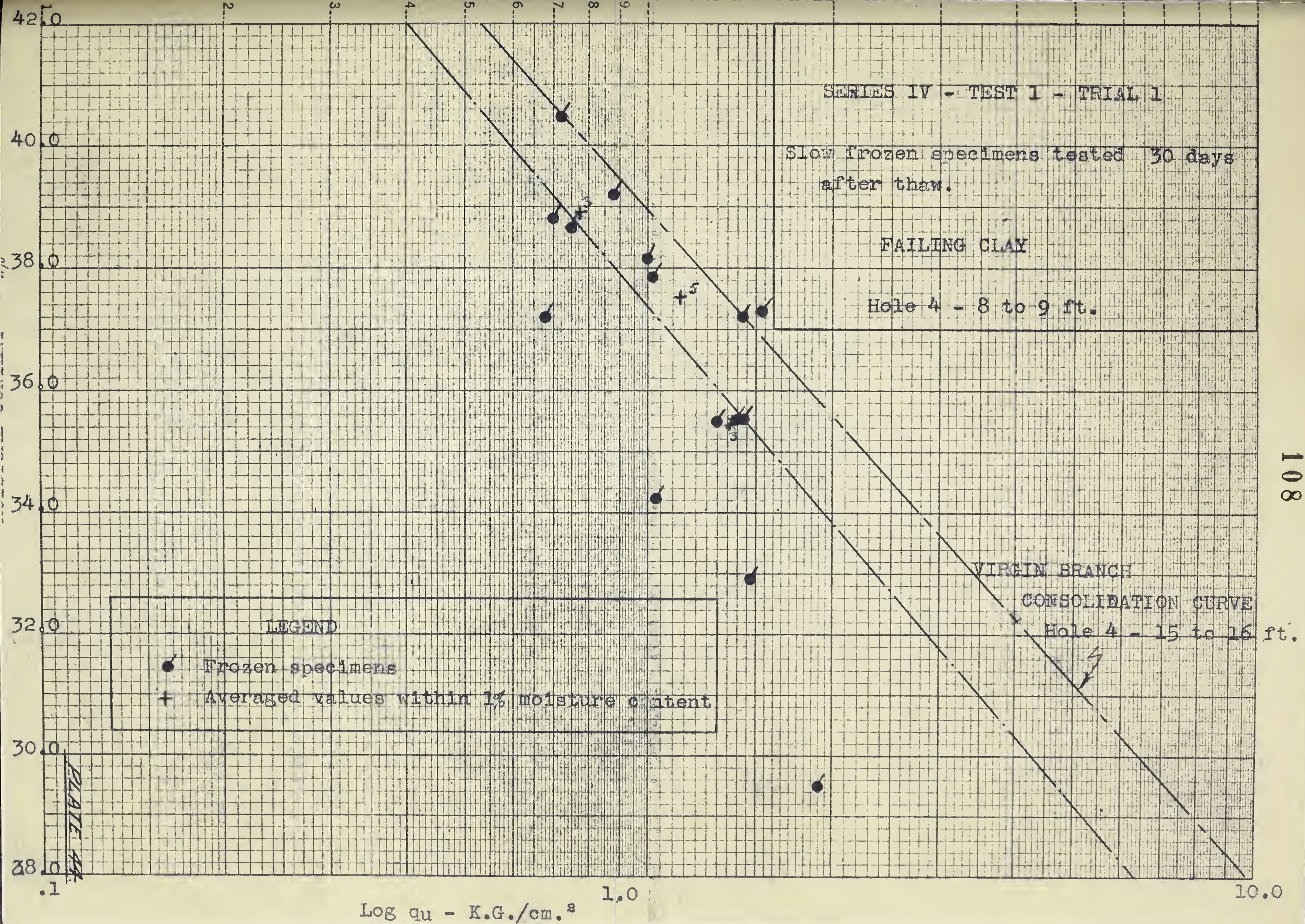

SERIES IV - TEST 1 - TRIAL 1
Slow frozen specimens tested 30 days after thaw.
FAILING CLAY
Hole 4 - 8 to 9 ft.
VIRGIN BRANCH
CONSOLIDATION CURVE
Hole 4 - 15 to 16 ft.
LEGEND
Frozen specimens
Averaged values within 1% moisture content
42.0
40.0
38.0
36.0
34.0
32.0
30.0
28.0
.1
2
3
4
5
6
7
8
9
1.0
10.0
Log qu - K.G./cm.²
3
5
3

PLATE 44

UNIVERSITY OF ALBERTA Dept. of Civil Engineering FREEZE - THAW EXPERIMENTS	Site: Failing Drilling Co. Location: 109 St & Kingsway, Edm. Hole No. 4 Depth: 8 ft- 9 ft. Moist. Cont 36.3 % Temp. 46 °F Date sampled: Nov 5th 54

Test Series: IV Test No: 2 Trial No: 1 Date of Test: Mar 18/55

OBJECT: To determine whether there is a recovery of strength in a clay 60 days following one cycle of slow freezing and thawing.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.70	69.9	28.4	41.5	.0005	16.6			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a) q_u	Layer (a) w%	Layer (b) q_u	Layer (b) w%	Layer (c) q_u	Layer (c) w%	Layer (c) q_u	Layer (c) w%
UNFROZEN		1								
		2								
		3								
		4								
(a)	1 cm.	5								
(b)		6								
		7								
(c)		8								
(d)		9								
		10								
		Av.								
	1.4 in.									
FROZEN		1	1.79	35.1	1.50	35.2	.91	32.95		
		2	1.23	33.9	1.11	38.15	1.08	36.7		
at ±25 °C		3	1.50	36.1	1.16	37.5	.78	37.55		
		4	1.20	37.6	1.00	37.1	1.05	37.6		
for 24 hrs	1 cm.	5			1.16	35.35	.72	37.9		
		6								
THAWED		7								
		8								
at ±35 °C		9								
for 24 hrs		10								
		Av.								
Tested 60 days after thaw	1.4 in.									

REMARKS
1. Series I, Test 3 is used as control for this test.
2. Specimen No. 4, layer (a) failed by fracturing.
3. Specimen No. 5, layer (c) split down the middle.

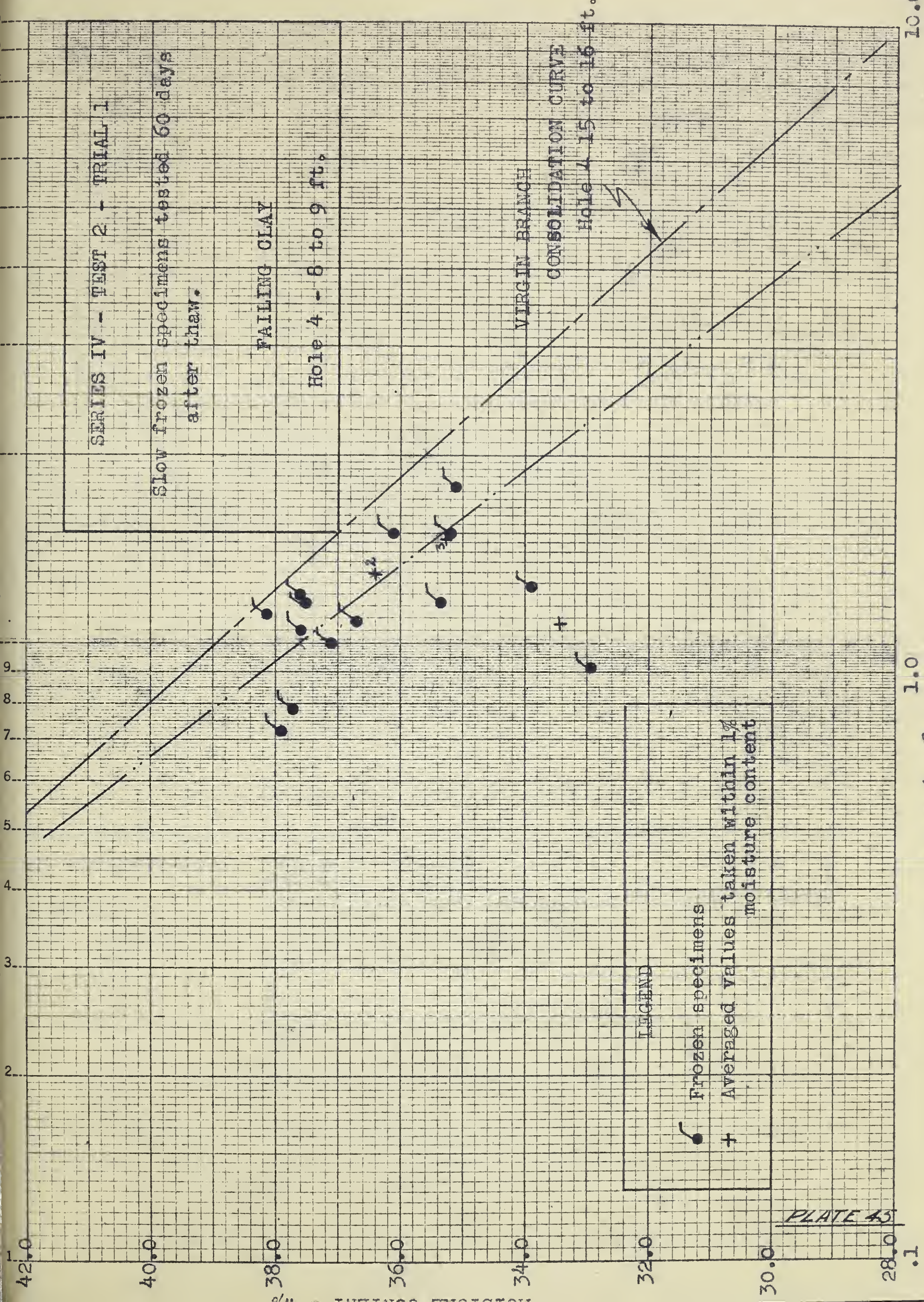

SERIES IV - TEST 2 - TRIAL 1
Slow frozen specimens tested 60 days after thaw.
FAILING CLAY
Hole 4 - 8 to 9 ft.
VIRGIN BRANCH
CONSOLIDATION CURVE
Hole 4 15 to 16 ft.
LEGEND
Frozen specimens
Averaged values taken within 1% moisture content
Log q_u - K.G./cm.²
42.0
40.0
38.0
36.0
34.0
32.0
30.0
28.0
.1
1.0
10.0
1
2
3
4
5
6
7
8
9
2
3

PLATE 45

UNIVERSITY OF ALBERTA
Dept. of Civil Engineering

FREEZE - THAW EXPERIMENTS

Site: Failing Drilling Co.
Location: 109 St & Kingsway, Edm.
Hole No. 4 Depth: 8ft-9ft
Moist. Cont. 36.3% Temp. 46°F
Date sampled: Nov 5th 54

Test Series: IV Test No: 3 Trial No: 1 Date of Test: Apr 25/55

OBJECT: To determine whether there is a recovery of strength of a clay 90 days following one cycle of slow freezing and thawing.

Classification Test Results	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
	2.70	69.9	28.4	41.5	.0005	16.6			CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (c)	
			q_u	W%	q_u	W%	q_u	W%	q_u	W%
UNFROZEN	1 cm.	1								
		2								
		3								
(a)		4								
(b)		5								
(c)		6								
(d)		7								
		8								
		9								
		10								
		Av.								
	1.4 in.					(discarded)				
FROZEN	1 cm.	1	1.55	36.3	1.28	32.2	1.03	36.8		
		2	1.82	35.9	.70	39.3	.73	38.5		
at ±25 °C		3	1.76	35.0	1.18	37.7	.60	38.3		
for 24 hrs		4	1.66	33.5	.76	38.6	.81	37.3		
THAWED		5	1.11	36.3	.77	38.9				
at ±35 °C		6			1.16	37.3				
for 24 hrs		7								
Tested 90 days after thaw		8								
		9								
		10								
		Av.								
	1.4 in.									

REMARKS
1. Series I, Test 3 is used as control for this test.
2. Specimen 5, layer (a) failed by fracturing.
3. Specimen 1, layer (c) showed signs of end restraint.
4. Specimen 1, layer (b) showed signs of end restraint and was discarded.

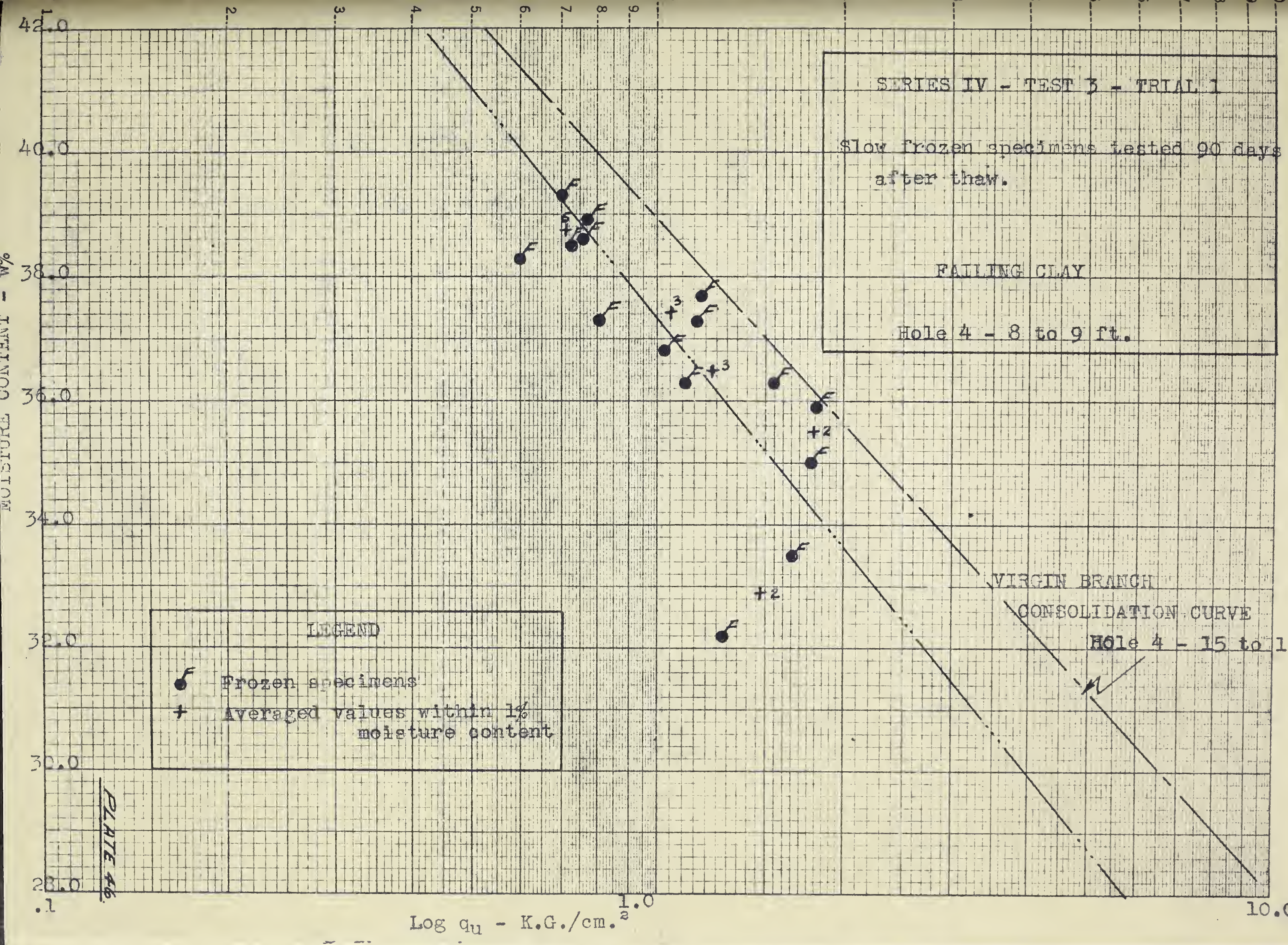
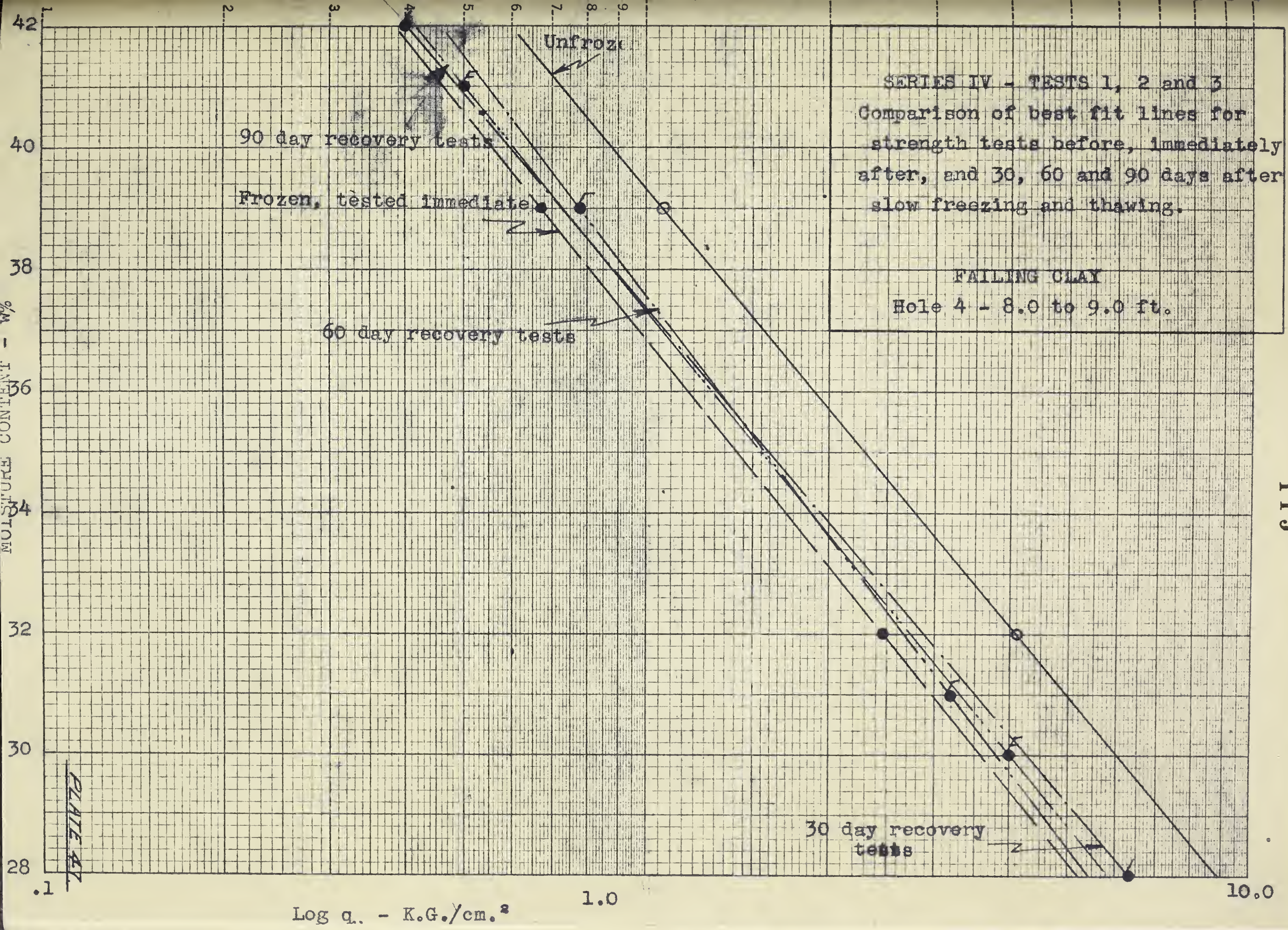

SERIES IV - TEST 3 - TRIAL 1
Slow frozen specimens tested 90 days after thaw.
FAILING CLAY
Hole 4 - 8 to 9 ft.
VIRGIN BRANCH CONSOLIDATION CURVE
Hole 4 - 15 to 16 f
LEGEND
Frozen specimens
+ Averaged values within 1% moisture content
F
+2
+3
42.0
40.0
38.0
36.0
34.0
32.0
30.0
28.0
.1
1.0
10.0
2
3
4
5
6
7
8
9
Log q_u - K.G./cm.^{2}
W% - MOISTURE CONTENT

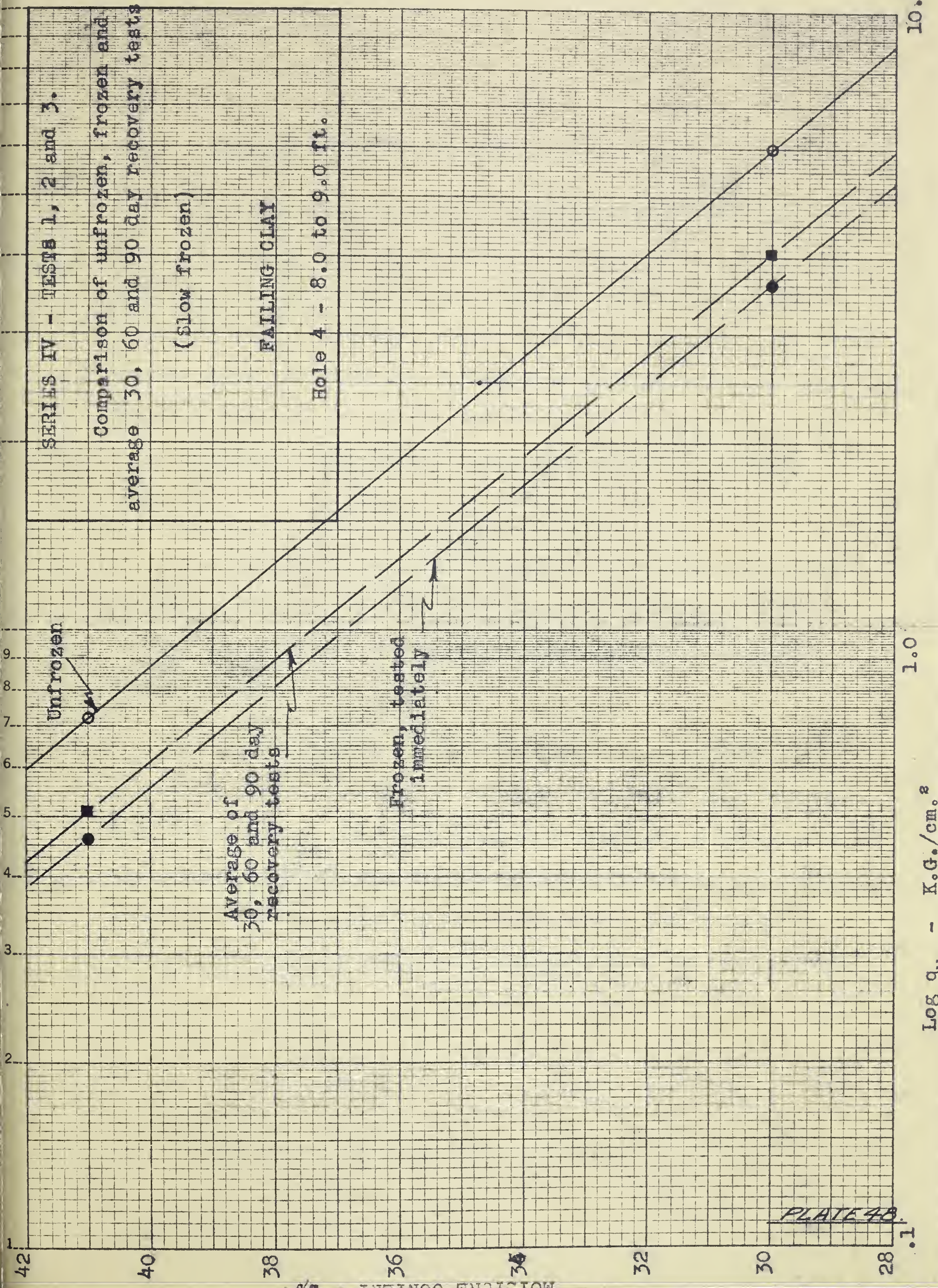
PLATE 46.



SERIES IV - TESTS 1, 2 and 3
Comparison of best fit lines for strength tests before, immediately after, and 30, 60 and 90 days after slow freezing and thawing.

FAILING CLAY
Hole 4 - 8.0 to 9.0 ft.

PLATE 47


SERIES IV - TESTS 1, 2 and 3.
Comparison of unfrozen, frozen and average 30, 60 and 90 day recovery tests.
(Slow frozen)
FAILING CLAY
Hole 4 - 8.0 to 9.0 ft.
Unfrozen
Average of 30, 60 and 90 day recovery tests
Frozen, tested immediately
Log q_u - K.G./cm.^{2}
10.0
1.0
.1
9
8
7
6
5
4
3
2
1
42
40
38
36
34
32
30
28
PLATE 48.

SERIES V - Triaxial Compression Tests.

Six 1.4 in. specimens were failed in triaxial compression, - three unfrozen and three frozen rapidly - and the Mohr circles obtained are plotted on Plate 50. With the exception of frozen specimen number three, which was failed with a lateral pressure of 6 K.G./cm.^{2}, the Mohr circles appear reasonable when difference in moisture content are taken into consideration. It appears from the circles obtained that the soil exhibits no loss of strength as a result of freezing when laterally confined.

A plot of moisture content versus log of deviator stress given on Plate 49 verifies the opinion that there was no loss of strength for this particular soil. This plot also substantiates the opinion that frozen specimen No. 3 was in error.

There is insufficient data from which to draw definite conclusions as regards the effect of confining pressure and it is suggested that further testing along these lines might constitute an extension to this project. However, from the data available it is noted that a line constructed through the average of the frozen points and through the average of the unfrozen points runs almost parallel to the consolidation curve.

UNIVERSITY OF ALBERTA
Dept. of Civil Engineering

FREEZE - THAW EXPERIMENTS

Site: Failing Drilling Co., Edm.
Location: 109 St & Kingsway
Hole No. 4 Depth: 10'6-11'6 and 14'6-15'6
Moist. Cont. ____ % Temp. ____ °F
Date sampled: ____ 5

Test Series: V Test No: 1 Trial No: 1 Date of Test: Feb 15/55

OBJECT: To determine the effects of confining pressure on the strength of clay following one cycle of rapid freezing and thawing.

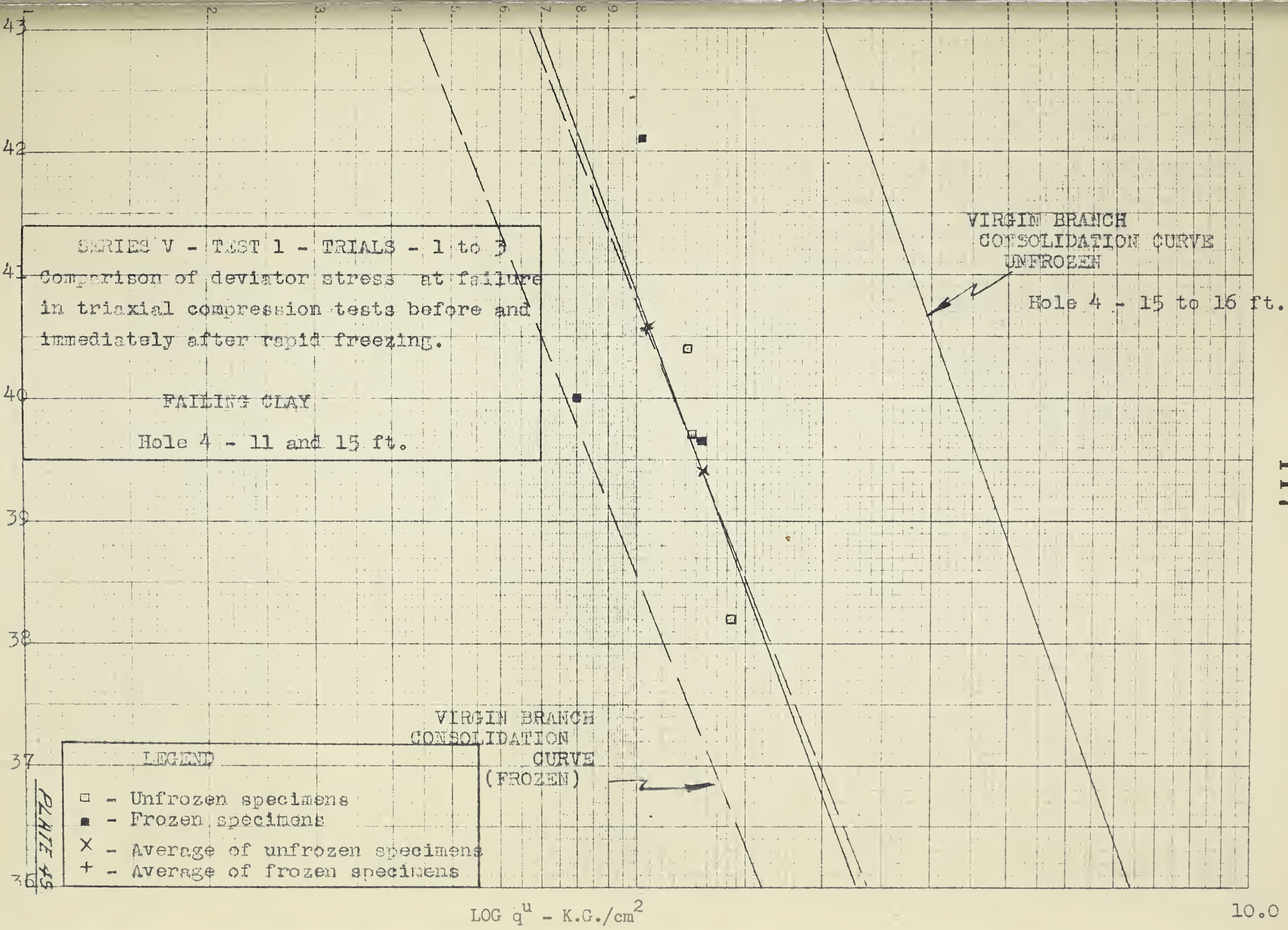
Classification	G_s	W_L	W_p	I_p	D_{10} mm.	U_c	C_c	k cm/sec	Casagrande Class.
Test Results 10'6-11'6	2.70	62.3	27.9	34.4					
14'6-15'6	2.70	64.5	26.6	37.9					CH

UNCONFINED COMPRESSIVE STRENGTH - K.G./cm.²

Test conditions	Size of Specimen		Layer (a)		Layer (b)		Layer (c)		Layer (c)	
			q_u	w%	q_u	w%	q_u	w%	q_u	w%
UNFROZEN		1	10 ft	to 11	14	to 15	10	to 11		
		2								
		3								
		4								
(a)	1 cm.	5								
(b)		6								
		7								
(c)		8								
(d)		9	W%	38.2		39.7		40.4		
		10	p_1	3.42		5.23		7.21		
		Av.	p_s	2.0		4.0		6.0		
	1.4 in.		(p_1-p_s)	1.42		1.23		1.21		
		1	14	to 15	10	to 11	14	to 15		
FROZEN		2								
		3								
at -30 °C		4								
for 24 hrs	1 cm.	5								
		6								
THAWED		7								
		8								
at +20 °C		9	W%	39.65		42.10		40.0		
		10	p_1	5.29		3.01		6.8		
for 24 hrs		Av.	p_s	4.0		2.0		6.0		
Tested immediately after thaw	1.4 in.		(p_1-p_3)	1.29		1.02		.80		

REMARKS

1. Unfrozen specimen of 10 to 11 ft depth (layer (c), first showed signs of failure at 110 gram load, at stress of 1.06 K.G./cm^2.

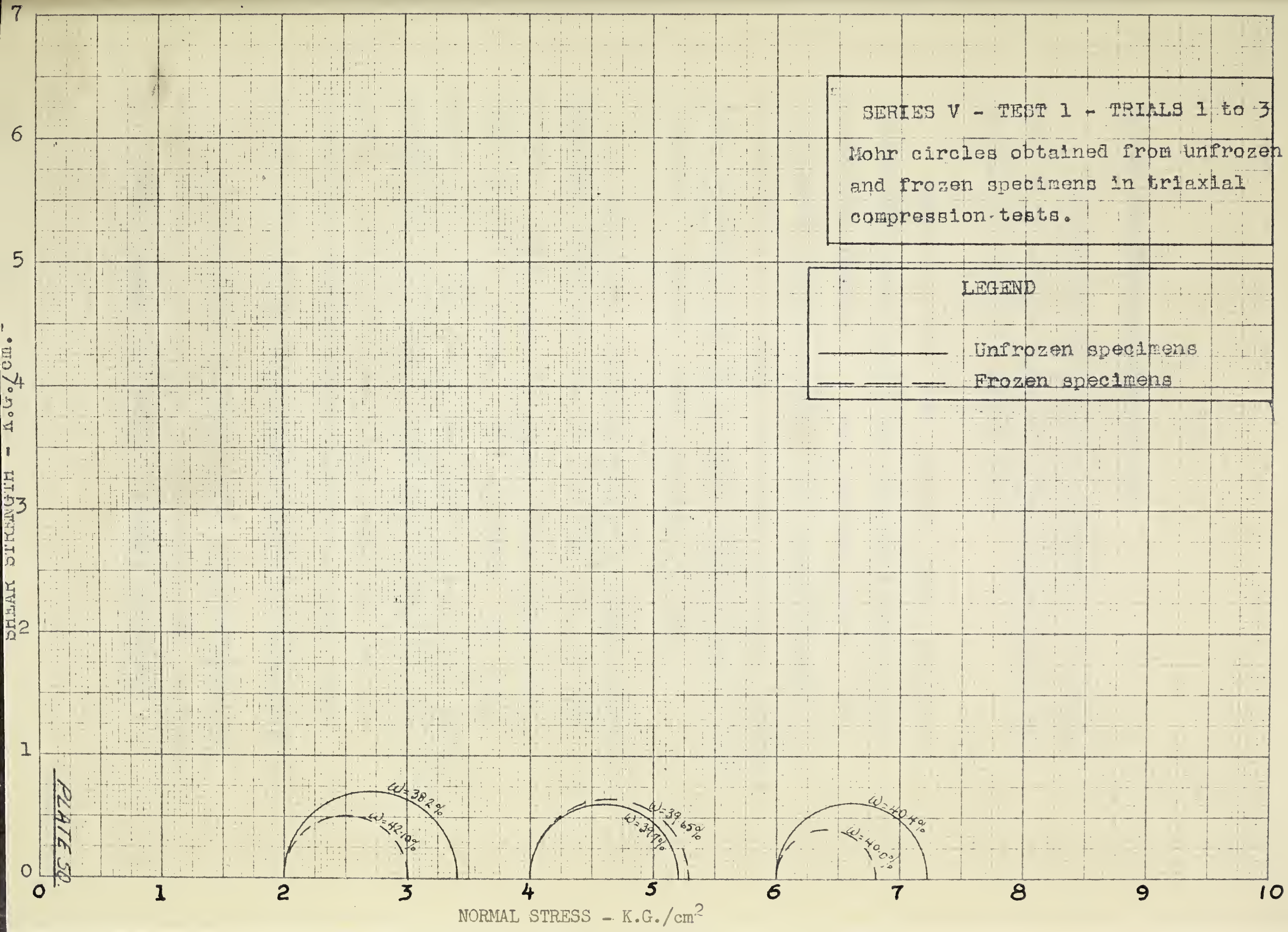


SERIES V - TEST 1 - TRIALS - 1 to 3
Comparison of deviator stress at failure in triaxial compression tests before and immediately after rapid freezing.

FAILING CLAY

Hole 4 - 11 and 15 ft.

PLATE 49



SERIES V - TEST 1 - TRIALS 1 to 3

Mohr circles obtained from unfrozen and frozen specimens in triaxial compression tests.

LEGEND

———	Unfrozen specimens
— — —	Frozen specimens

PLATE 50.

General Conclusions from Strength Tests

Results from Series I showed that there was a definite reduction in strength of clay as a result of both rapid and slow freezing and thawing in a closed system.

Results of these tests indicated a greater reduction in strength after rapid freezing than after slow freezing; in the case of rapid freezing at an average moisture content of $35^{\circ}/o$, the reduction amounted to $38.0^{\circ}/o$ of the unfrozen strength; in the case of the slow freezing reduction was approximately $33.0^{\circ}/o$. These are average values for all samples tested in Series I. The reduction for individual samples varies with the sample.

Prof. Yoder's report published by the Highway Research Board (Ref. 9) indicates that a clay soil and Silty clay both suffered a strength loss of 82 percent of the initial value after one cycle of freezing at 24°F and thawing. The results further indicate that the clay lost only 74 percent of its initial strength after being frozen at -18°F while the silty clay lost 97 percent. It should of course be borne in mind that tests were carried out on remolded specimens compacted to Proctor density and that the specimens were frozen and thawed in an open system.

In the case of the McKernan silty clay insufficient test data is available from which to conclude whether there was or was not a loss of strength as a result of freezing and thawing.

Confined tests on shale showed no definite loss of strength with freezing. Additional fracturing and segregation of the material is however evident which appears to cause the shale to assume the properties of a granular material, with a subsequent increase of strength at higher lateral pressure. Additional cycles of freezing would likely aggravate this condition until a point would be reached where the material, upon addition of water, would exhibit the properties of a highly plastic clay. In this state, the material could be expected to show a loss of strength with freezing with no addition of moisture.

It could be concluded from the results of Series III that there is a definite recovery of strength of a clay with time following rapid freezing and thawing. The percentage of strength recovered and/or the rate of recovery appear to vary widely with samples tested. In one case there was almost complete recovery after 30 days while in another the recovery of strength appeared to continue beyond 90 days.

Test results showed that there was also a recovery of strength following a slow freeze; but for the one sample tested the percentage of recovery appears to be less than in the case of rapid freezing.

From the limited test data available it appears that there may not be a reduction in strength following rapid freezing as exhibited in triaxial compression tests.

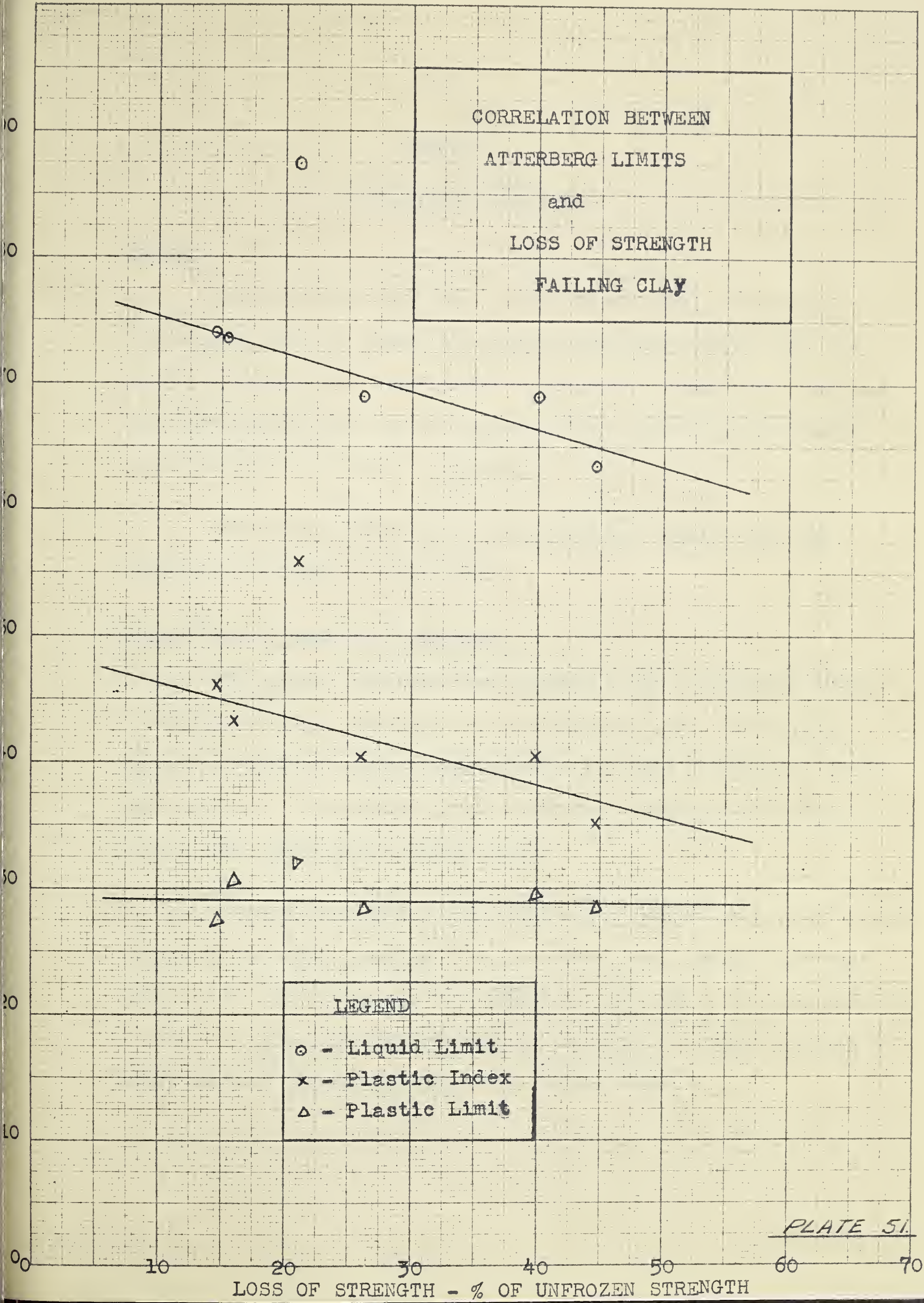

CORRELATION BETWEEN
ATTERBERG LIMITS
and
LOSS OF STRENGTH
FAILING CLAY
LEGEND
⊙ - Liquid Limit
× - Plastic Index
△ - Plastic Limit
0
10
20
30
40
50
60
70
LOSS OF STRENGTH - % OF UNFROZEN STRENGTH

PLATE 51.

CHAPTER 7

CONSOLIDATION TESTS

General

Consolidation tests were carried out on frozen and unfrozen specimens in order to compare the virgin consolidation curves with the moisture content versus logarithm of compressive strength curves obtained from strength tests as well as to determine any change in consolidation characteristics as a result of freezing.

Tests were carried out on three different samples which are referred to as Tests 1, 2 and 3 below.

Preparation and Handling of Specimens

The unfrozen specimens were prepared in the conventional manner according to A.S.T.M. Standards for consolidation tests. At the same time, corresponding specimens were prepared for rapid freezing and others were remolded. The specimens for freezing were sealed in waxed metal containers while still in the cutting rings.

In the case of Test 3, one specimen was remolded at natural moisture content by thoroughly kneading it in the moist room. The soil was alternately rolled into a one-half inch thread and then into a ball approximately 60 times. A second specimen was remolded at moisture content in excess of liquid limit by thorough mixing in the electric mixer.

Test Procedure

Conventional consolidation test procedure was followed which is given as Appendix "D" to this report.

Data and Results

Test 1 and 2 - Failing Clay - Hole 2 at 10 -6" and Hole 3 at 10 -6" respectively.

The frozen specimen of Test 1 was ruined during test. The compression versus pressure curve showed considerable discontinuity which probably indicated slippage of the dial during the test. This specimen was therefore discarded.

There was only sufficient undisturbed material in sample 2 to provide one specimen. This specimen was tested following rapid freezing and thawing. The results of this trial along with a remolded specimen of the same sample are presented with the results of Test 1 on Plate 55.

Since these tests were run on two different samples, no conclusions regarding the effect of freezing on consolidation characteristics can be arrived at. The compression curves have however been used for comparison with strength curves.

MTL-4

MATERIALS TESTING LABORATORIES LTD.

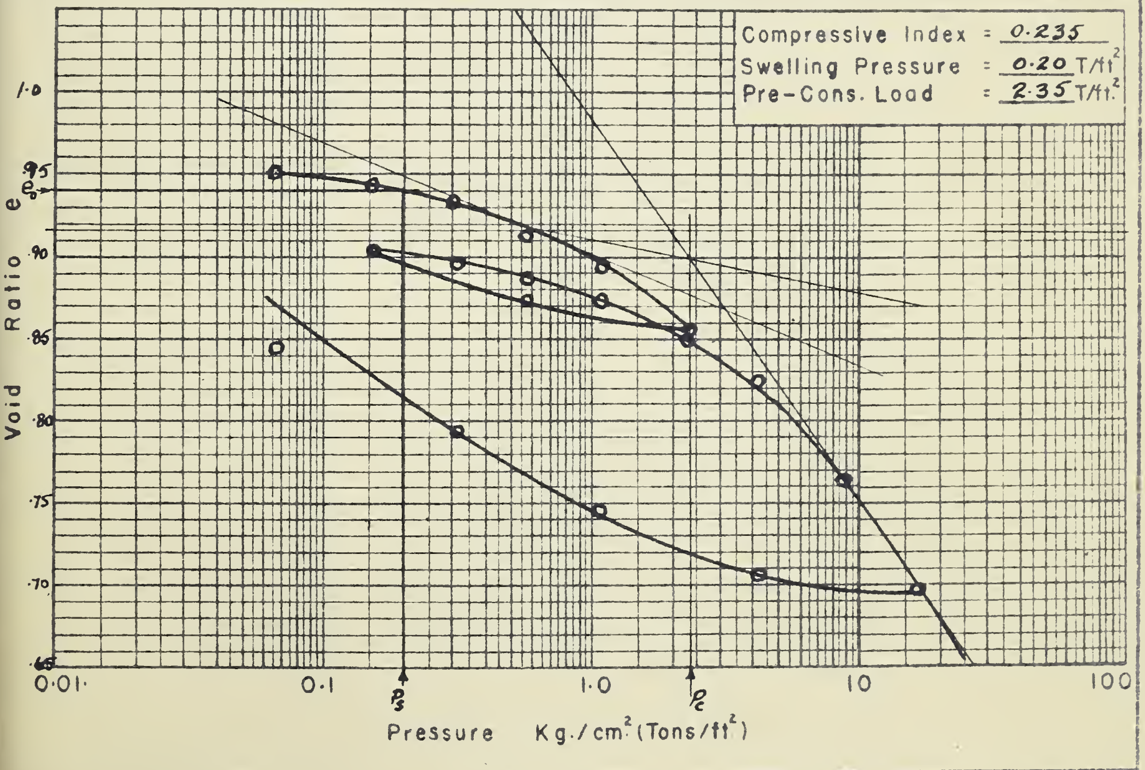
CONSOLIDATION RESULTS

PROJECT CE 106
LAB. ORDER
SAMPLE CLAY
LOCATION
HOLE 2 DEPTH 10⁶– 11⁶
TECHNICIAN CURTIS + DICKSON DATE

Specific Gravity of Soil Solids G_s = 2·70 Height of Soil Solids H_s = ·501 ins
Void Ratio e (End) = ·845
Void Ratio e(Start) = ·894/S = ·940 S = .95
Void Ratio e (Start Dimensions) = ·765

$e(\text{End}) = W\%(\text{End}) \times G_s$ $H_s = \left(\frac{\text{Wt}\cdot\text{Soil}}{G_s \times \text{Area} \times 2{\cdot}54}\right)\text{ins.}$ $e = \text{previous } e \pm \frac{\text{Def'l.}}{H_s}$

Time Interval	Load on Pan (gms)	Corr. Dial Reading (ins.)	Deflection (ins.)	Deflection / H_s	Void Ratio e	Pressure Kg/cm² = T/ft²
—	0	·8660	—	—	·940	—
1101 MINS.	20	·8710	·0050	+·010	·950	·0683
90	50	·8692	·0032	+·006	·946	·1565
390	100	·8630	·0030	–·006	·934	·302
360	200	·8513	·0147	–·029	·911	·593
240	400	·8389	·0271	·054	·896	1·175
720	800	·8236	·0424	·085	·855	2·335
160	200	·8323	·0337	·067	·873	·593
3284	50	·8474	·0186	·037	·903	·1565
60	100	·8454	·0206	·041	·899	·302
67	200	·8401	·0259	·052	·888	·593
720	400	·8319	·0341	·068	·872	1·175
2760	800	·8220	·0440	·088	·852	2·335
306	1500	·8074	·0568	·117	·823	4·37



Compressive Index = 0·235
Swelling Pressure = 0·20 T/ft²
Pre-Cons. Load = 2·35 T/ft²

PLATE 52.

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

CONSOLIDATION RESULTS

PROJECT THESIS.
SITE FAILING DRILLING CO.
SAMPLE UNDISTURBED, QUICK FROZEN.
LOCATION 109th st & 111th AVE.
HOLE 3 DEPTH 10.5'-11.5'.
TECHNICIAN CURTIS & DICKSON. DATE MARCH 25/55.

Specific Gravity of Soil Solids G_s = 2.73 Height of Soil Solids H_s = .412 ins.
Void Ratio e (End) = .985
Void Ratio e(Start) = 1.201 $S_s = 98.5\%$
Void Ratio e (Start Dimensions) = 1.18

$e(\text{End}) = W\%(\text{End}) \times G_s$ $H_s = \left(\frac{\text{Wt. Soil}}{G_s \times \text{Area} \times 2.54}\right)$ ins. $e = \text{previous } e \pm \frac{\text{Def'l.}}{H_s}$

Time Interval	Load on Pan (gms)	Corr. Dial Reading (ins.)	Deflection (ins.)	Deflection / H_s	Void Ratio e	Pressure Kg/cm.^{2} = T/ft.^{2}
0 min.	0	.8485	0		1.201	.016
1278 m.	50	.8495	+.0010	+.0024	1.203	.157
1470 m.	100	.8465	−.0020	−.0048	1.196	.303
1320 m.	200	.8389	.0096	.0233	1.178	.594
1560 m.	400	.8214	.0271	.0658	1.135	1.174
1380 m.	800	.7950	.0535	.130	1.071	2.337
600 m	1500	.7705	.0780	.189	1.012	4.377
780 m	3000	.7288	.1197	.291	.910	8.74
1740 m	6000	.6819	.1666	.404	.797	17.47
1710 m	12000	.6320	.2165	.526	.675	34.96
750 m	0	.7592	.0893	.216	.985	.016

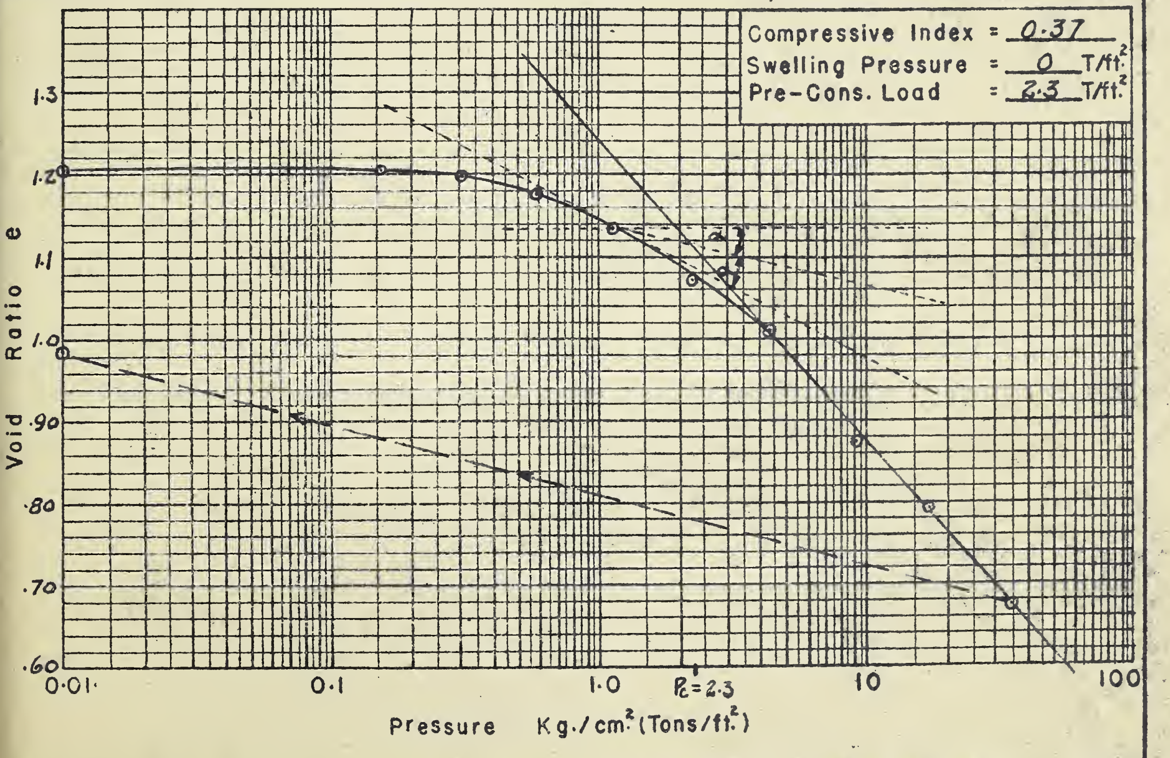

Compressive Index = 0.37
Swelling Pressure = 0 T/ft.^{2}
Pre-Cons. Load = 2.3 T/ft.^{2}

PLATE 53.

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

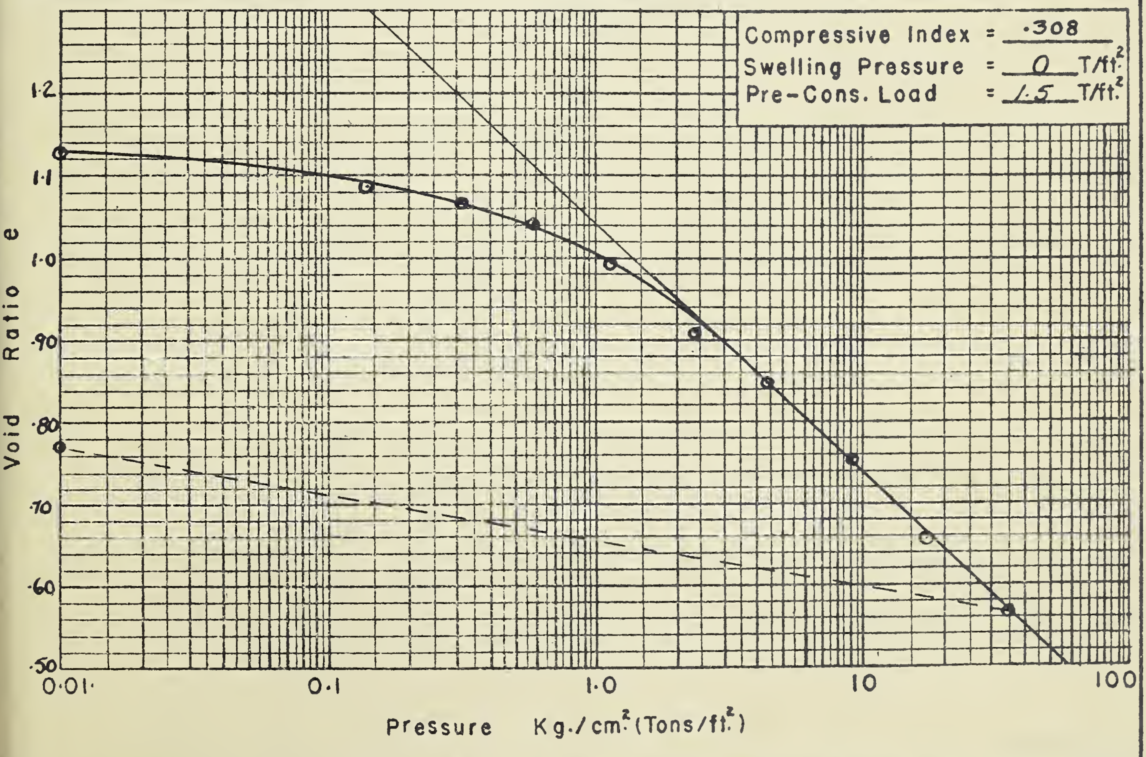
CONSOLIDATION RESULTS

PROJECT *THESIS.*
SITE *FAILING DRILLING CO.*
SAMPLE *QUICK FREEZE, REMOULDED.*
LOCATION *109th st & 111 Ave. EDMONTON.*
HOLE *3.* DEPTH *10.5'-11.5'.*
TECHNICIAN *CURTIS & DICKSON.* DATE *MARCH 25/55.*

Specific Gravity of Soil Solids G_s = *2.73* Height of Soil Solids H_s = *.425* ins.
Void Ratio e (End) = *.773*
Void Ratio e(Start) = *1.091/S = 1.105* *S = 98%*
Void Ratio e (Start Dimensions) = *1.105*

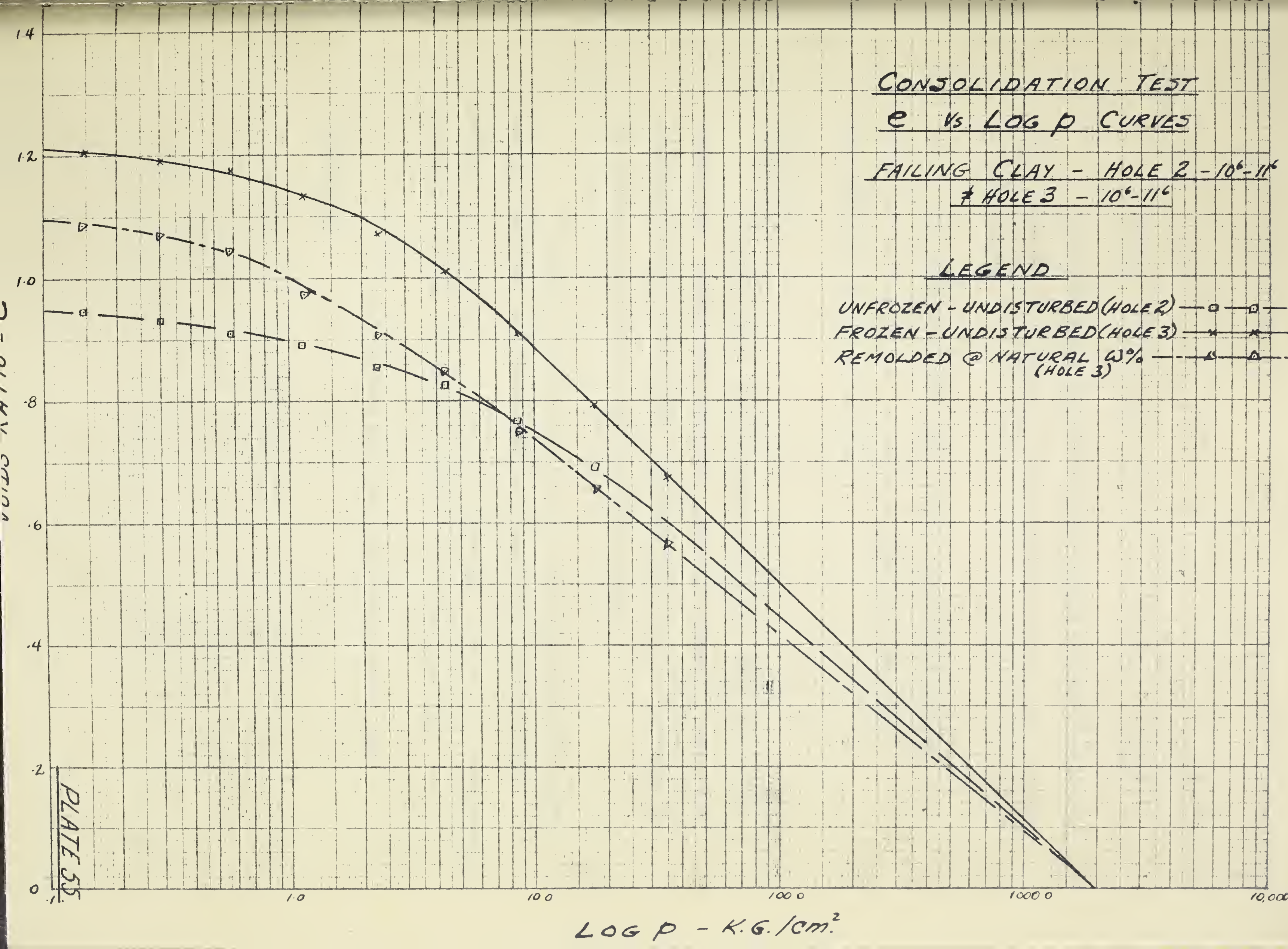
e(End) = W%(End) x G_s $H_s = \left(\frac{\text{Wt. Soil}}{G_s \times \text{Area} \times 2.54}\right)$ ins. e = previous e $\pm \frac{\text{Def'l.}}{H_s}$

Time Interval	Load on Pan (gms)	Corr. Dial Reading (ins.)	Deflection (ins.)	Deflection / H_s	Void Ratio e	Pressure Kg/cm² = T/ft²
0 min.	gms.	.8414	0	0	1.1271	.0110
1281 m.	50	.8243	.0171	-.0402	1.0869	.1463
1380 m.	100	.8182	.0232	-.0546	1.0725	.302
1320 m.	200	.8051	.0363	-.0854	1.0417	.583
1560 m.	400	.7820	.0594	-.1395	0.9876	1.172
1320 m	800	.7522	.0892	-.2200	0.9071	2.335
600 m.	1500	.7250	.1164	-.2759	0.8512	4.38
780 m	3000	.6828	.1586	-.3738	0.7533	8.74
1740 m	6000	.6425	.1989	-.4680	0.6591	17.47
1710 m.	12000	.6032	.2382	-.5610	0.5661	34.96
770 m	0	.6910	.1504	-.3541	0.7730	.0110



Compressive Index = *.308*
Swelling Pressure = *0* T/ft.²
Pre-Cons. Load = *1.5* T/ft.²

PLATE 54.



CONSOLIDATION TEST
e Vs. LOG p CURVES

FAILING CLAY - HOLE 2 - 10'-11'
" HOLE 3 - 10'-11'

PLATE 55.

Test 3 - Failing Clay - Hole 4, 15 to 16 ft.

Specimens were obtained from this sample unfrozen undisturbed, frozen undisturbed, remolded at natural moisture content and remolded at liquid limit.

The resulting voids ratio versus logarithm of pressure curves for all four trials are given on Plate 60. One set of time curves are given on Plate 61.

Conclusions

It would appear from the results of Test 3 above that rapid freezing and thawing does effect the compressive index and causes the compression curve to run almost coincident with the curve obtained for the specimen remolded at natural moisture content.

It appears from Plate 61 on which are plotted the time curves for the 6000 gram load increment that there is very little if any change in permeability as a result of freezing. The rate of consolidation is nearly the same for both the unfrozen and frozen specimen.

It is felt that more tests are required to verify these conclusions.

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
CONSOLIDATION RESULTS

PROJECT THESIS
SITE FAILING
SAMPLE LAYER b. Undisturbed
LOCATION
HOLE 4 DEPTH 15'-16'
TECHNICIAN C & D DATE APRIL 26/55

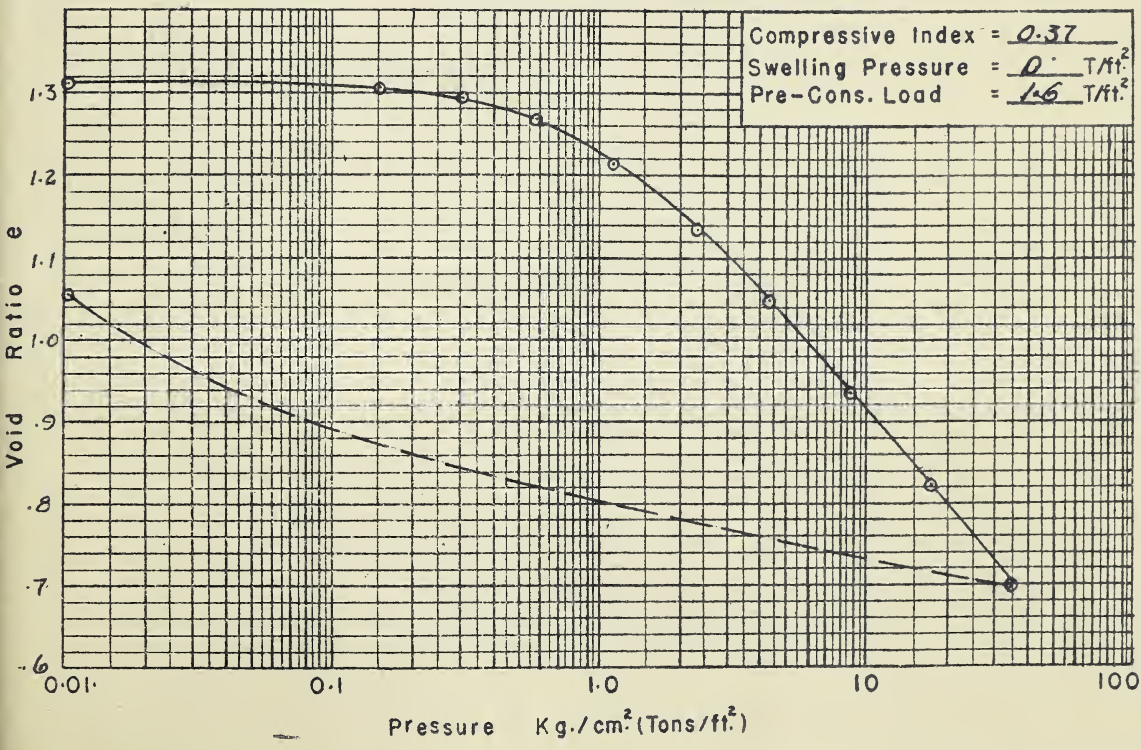
Specific Gravity of Soil Solids G_s = 2.70 Height of Soil Solids H_s = .3962 ins.
Void Ratio e (End) = 1.055
Void Ratio e(Start) = 1.312
Void Ratio e (Start Dimensions) = 1.27

$e(End) = W\%(End) \times G_s$

$H_s = \left(\frac{Wt \cdot Soil}{G_s \times Area \times 2.54}\right)$ ins.

$e = \text{previous } e \pm \frac{Def'l.}{H_s}$

Time Interval	Load on Pan (gms)	Corr. Dial Reading (ins.)	Deflection (ins.)	$\frac{Deflection}{H_s}$	Void Ratio e	Pressure Kg/cm² = T/ft²
minutes 0	0	.8085	—		1.312	.0105
1050	50	.8052	.0033	.0083	1.304	.156
1450	100	.8018	.0067	.0169	1.295	.301
1440	200	.7908	.0177	.0446	1.267	.584
1470	400	.7696	.0389	.0981	1.214	1.173
1800	800	.7386	.0699	.1763	1.136	2.335
1440	1500	.7036	.1049	.2643	1.048	4.375
1330	3000	.6597	.1488	.3750	.937	8.73
1130	6000	.6137	.1948	.4910	.821	17.44
1380	12000	.5648	.2437	.6140	.698	34.45
1800	0	.7067	.1018	.2565	1.055	.0105



Compressive Index = 0.37
Swelling Pressure = 0 T/ft²
Pre-Cons. Load = 1.6 T/ft²

PLATE 56.

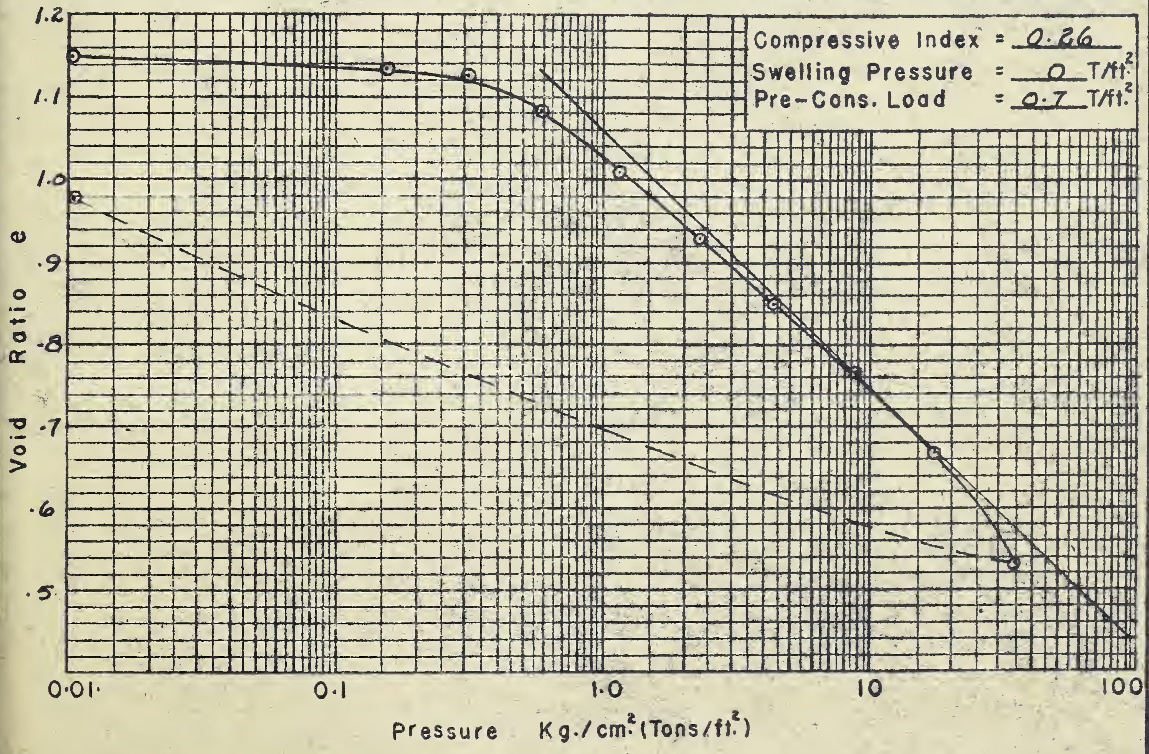
UNIVERSITY of ALBERTA
DEPT of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
CONSOLIDATION RESULTS

PROJECT *THESIS*
SITE *FAILING*
SAMPLE *(Layer 'd': Undisturbed-Frozen)*
LOCATION
HOLE *4* DEPTH *15'-16'*
TECHNICIAN *Curtis & Dickson* DATE *APRIL 26/55*

Specific Gravity of Soil Solids G_s = *2.70* Height of Soil Solids H_s = *.471* ins.
Void Ratio e (End) = *.980*
Void Ratio e(Start) = *1.151*
Void Ratio e (Start Dimensions) = *0.91*

e(End) = W%(End) x G_s $H_s = \left(\frac{\text{Wt. Soil}}{G_s \times \text{Area} \times 2.54}\right)$ ins. e = previous e ± $\frac{\text{Def'l.}}{H_s}$

Time Interval	Load on Pan (gms)	Corr. Dial Reading (ins.)	Deflection (ins.)	$\frac{\text{Deflection}}{H_s}$	Void Ratio e	Pressure Kg/cm² = T/ft²
0	0	.8748	—		1.151	.0105
1310	50	.8670	.0078	.0166	1.134	.159
1510	100	.8618	.0130	.0276	1.123	.304
1375	200	.8418	.0330	.0702	1.081	.595
1675	400	.8092	.0656	.1394	1.012	1.175
1540	800	.7714	.1034	.220	.931	2.340
1740	1500	.7349	.1399	.297	.854	4.37
3960	3000	.6930	.1818	.386	.765	8.73
1440	6000	.6450	.2298	.488	.663	17.44
1525	12000	.5840	.2908	.618	.533	34.45
2070	0	.7944	.0804	.171	.980	.0105



Compressive Index = *0.26*
Swelling Pressure = *0* T/ft²
Pre-Cons. Load = *0.7* T/ft²

PLATE 57

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

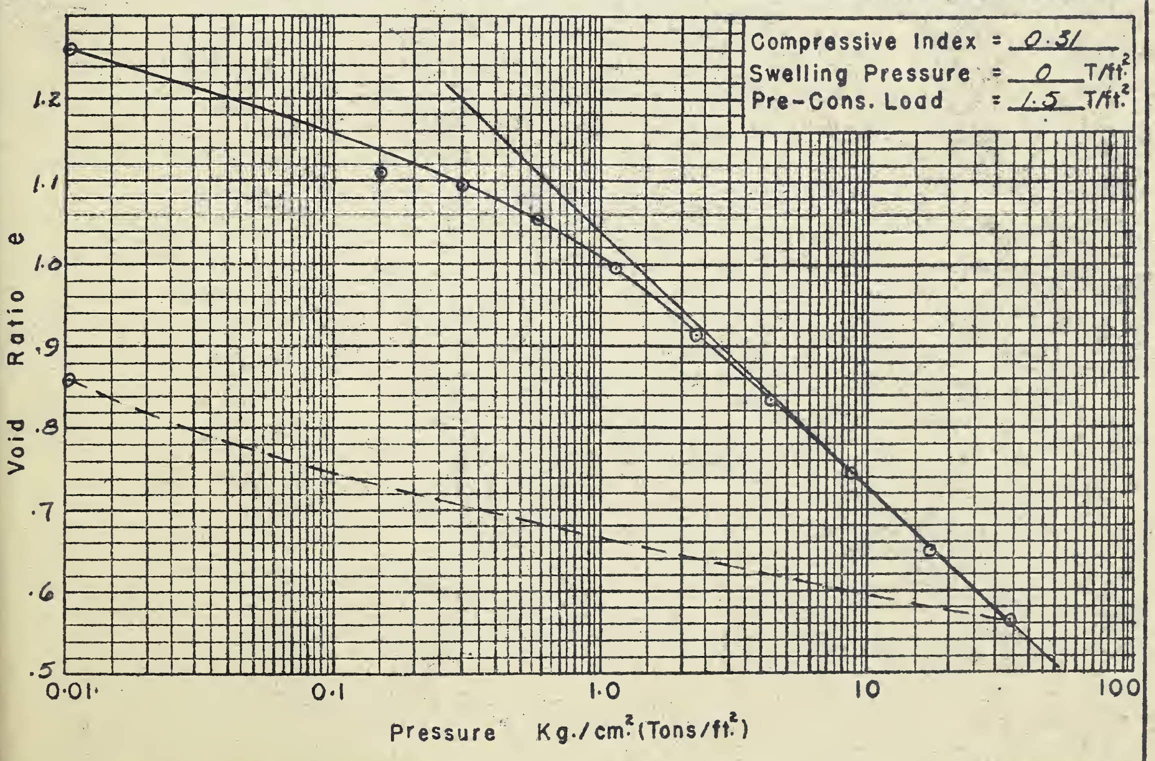
CONSOLIDATION RESULTS

PROJECT THESIS
SITE FAILING
SAMPLE Layer 'a' Remolded @ natural moisture content.
LOCATION
HOLE 4 DEPTH 15'-16'
TECHNICIAN Curtis & Nickson DATE APRIL 26/55

Specific Gravity of Soil Solids G_s = 2.70 Height of Soil Solids H_s = .428 ins.
Void Ratio e (End) = .860
Void Ratio e (Start) = 1.126
Void Ratio e (Start Dimensions) = 1.11

$e(\text{End}) = W\%(\text{End}) \times G_s$ $H_s = \left(\frac{\text{Wt. Soil}}{G_s \times \text{Area} \times 2.54}\right)$ ins. $e = \text{previous } e \pm \frac{\text{Def'l.}}{H_s}$

Time Interval	Load on Pan (gms)	Corr. Dial Reading (ins.)	Deflection (ins.)	Deflection / H_s	Void Ratio e	Pressure Kg/cm² = T/ft²
0	0	.7898	—		1.126	.0105
1050	50	.7820	.0078	.0182	1.108	.156
1450	100	.7763	.0135	.0315	1.094	.301
1440	200	.7593	.0305	.0712	1.055	.584
1470	400	.7325	.0573	.1338	.992	1.173
1800	800	.6986	.0912	.2130	.913	2.335
1440	1500	.6650	.1248	.2910	.835	4.375
1330	3000	.6259	.1639	.3820	.744	8.73
1130	6000	.5865	.2033	.4740	.652	17.44
1380	12000	.5477	.2421	.5650	.561	34.45
1800	0	.6758	.1140	.2660	.860	.0105



Compressive Index = 0.31
Swelling Pressure = 0 T/ft²
Pre-Cons. Load = 1.5 T/ft²

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

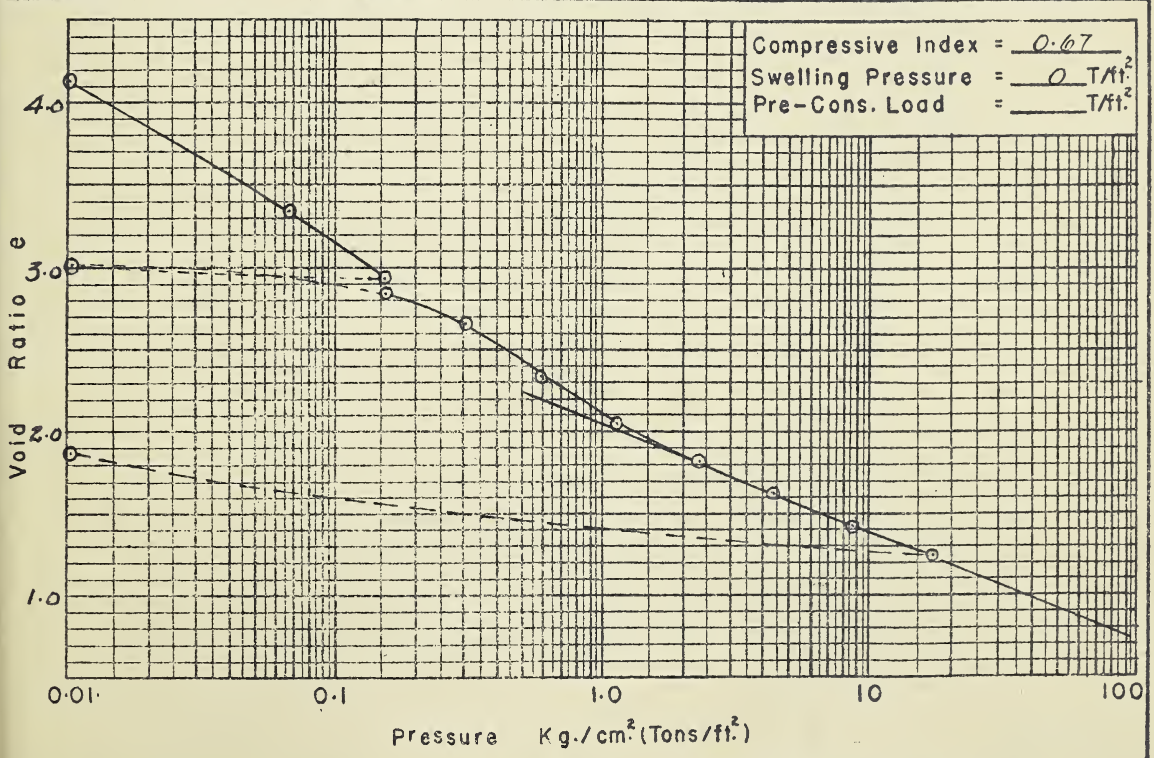
CONSOLIDATION RESULTS

PROJECT THESIS
SITE
SAMPLE FAILING CLAY
LOCATION (REMOLDED AT LIQUID LIMIT)
HOLE 4 DEPTH 15'-16'
TECHNICIAN Curtis & Dickson DATE APRIL/55

Specific Gravity of Soil Solids G_s = 2·70 Height of Soil Solids H_s = ·1615 ins.
Void Ratio e (End) = 1·898
Void Ratio e(Start) = 4·858
Void Ratio e (Start Dimensions) = 4·57

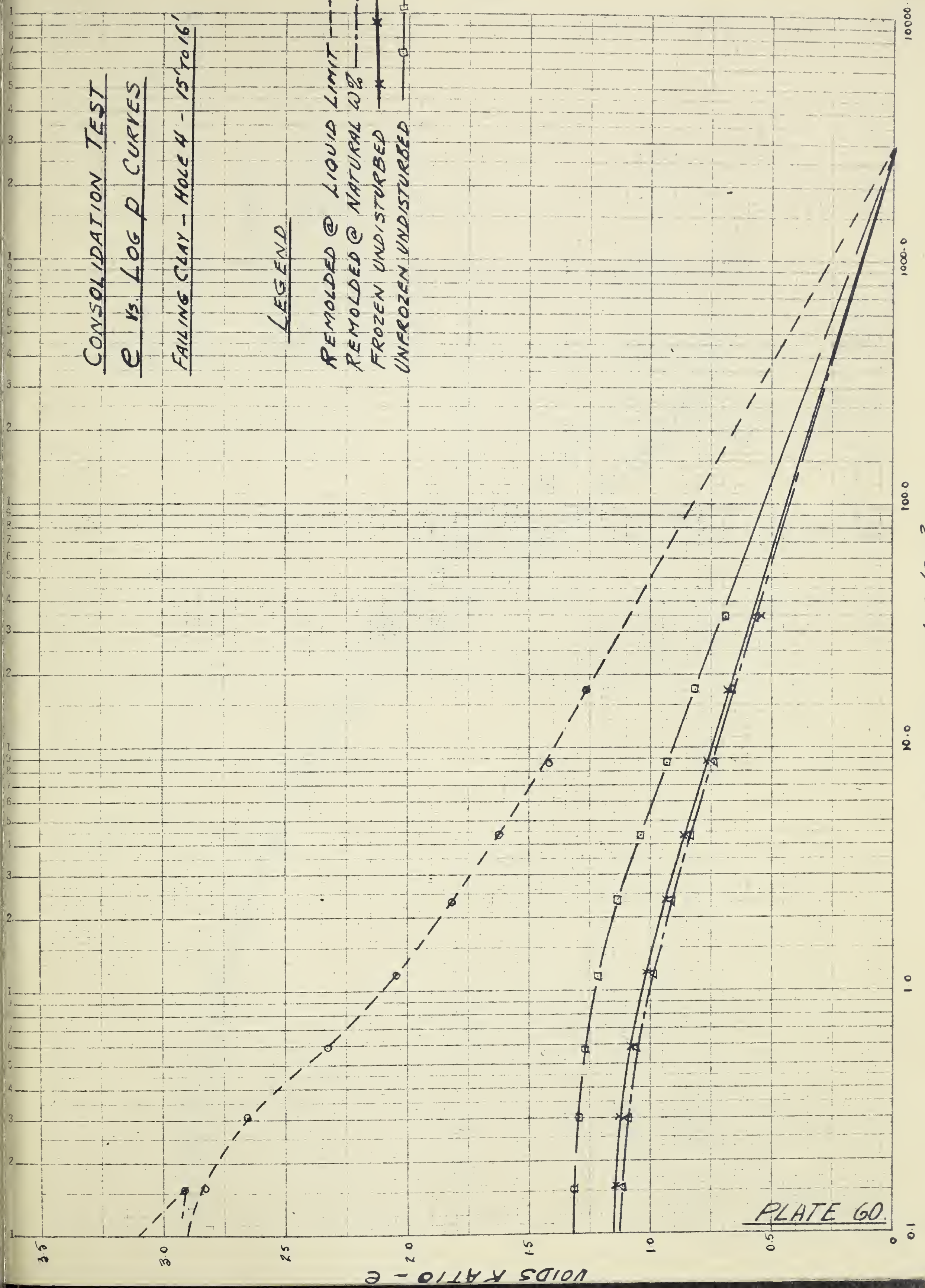
$e(\text{End}) = W\%(\text{End}) \times G_s$ $H_s = \left(\frac{\text{Wt. Soil}}{G_s \times \text{Area} \times 2{\cdot}54}\right)$ ins. $e = \text{previous } e \pm \frac{\text{Def'l.}}{H_s}$

Time Interval	Load on Pan (gms)	Corr. Dial Reading (ins.)	Deflection (ins.)	$\frac{\text{Deflection}}{H_s}$	Void Ratio e	Pressure Kg/cm² = T/ft²
(minutes)		.8280			4·858	
1095	0	.7125	·1155	·716	4·142	·0105
1440	20	.5845	·2435	1·510	3·348	·0688
1440	50	.5163	·3117	1·930	2·928	·156
1715	0	.5288	·2992	1·853	3·005	·0105
1030	50	.5045	·3235	2·005	2·853	·159
1420	100	.4718	·3562	2·206	2·652	·304
1500	200	.4192	·4088	2·530	2·328	·595
1375	400	·3750	·4530	2·805	2·053	1·175
1500	800	·3350	·4930	3·050	1·808	2·340
1540	1500	·3044	·5236	3·240	1·618	4·37
3960	3000	·2722	·5558	3·440	1·418	8·73
1440	6000	·2465	·5815	3·600	1·258	17·44
1525	0	·3507	·4773	2·960	1·898	·0105



Compressive Index = 0·67
Swelling Pressure = 0 T/ft²
Pre-Cons. Load = T/ft²

PLATE 59.



CONSOLIDATION TEST

e vs. LOG p CURVES

FAILING CLAY - HOLE 4 - 15' TO 16'

PLATE 60.

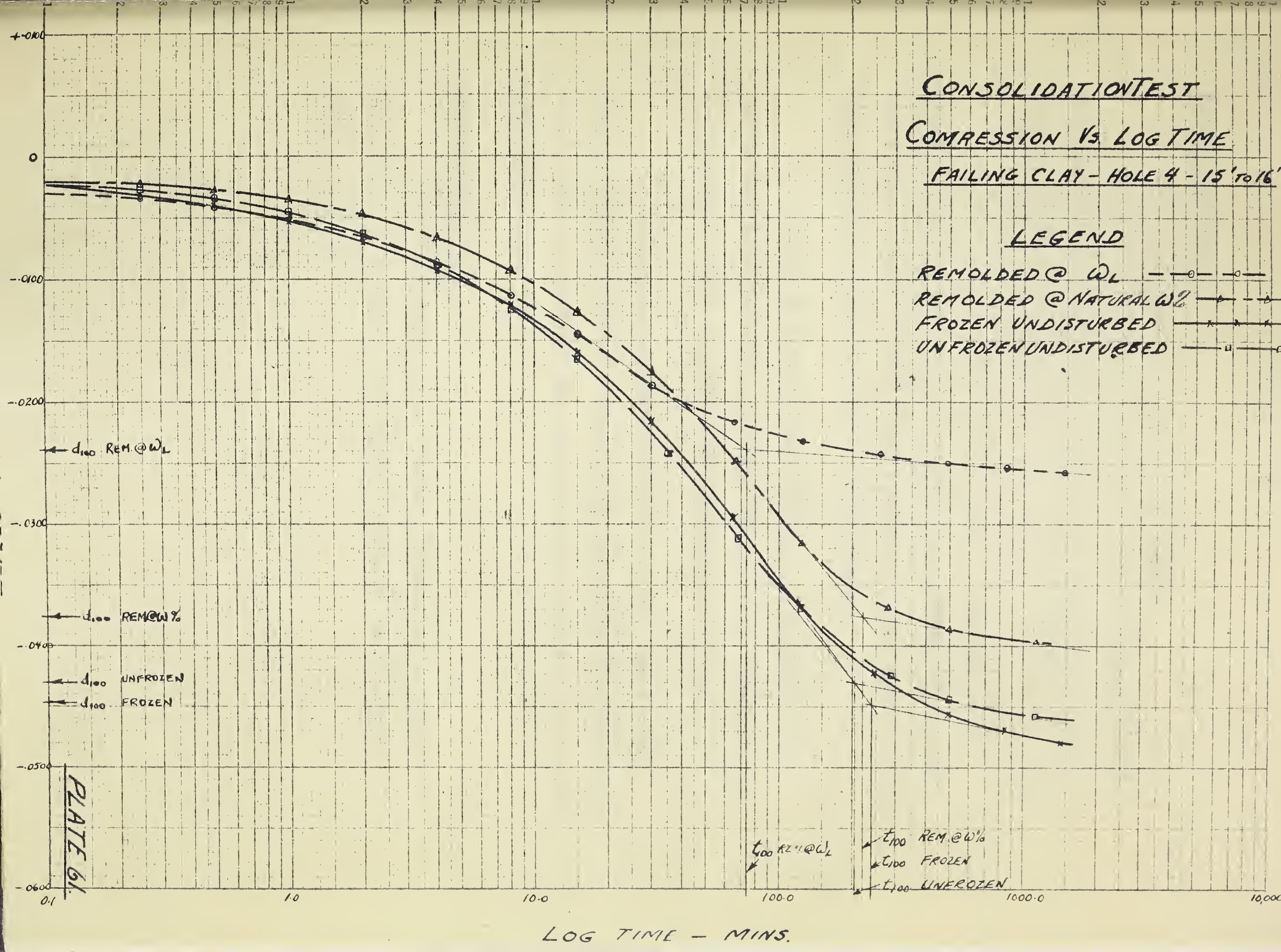

CONSOLIDATION TEST
COMPRESSION Vs LOG TIME
FAILING CLAY - HOLE 4 - 15' TO 16'
LEGEND
REMOLDED @ ω_L
REMOLDED @ NATURAL ω%
FROZEN UNDISTURBED
UNFROZEN UNDISTURBED
+.0100
0
-.0100
-.0200
-.0300
-.0400
-.0500
-.0600
0.1
1.0
10.0
100.0
1000.0
10,000.0
LOG TIME - MINS.
d_{100} REM. @ ω_L
d_{100} REM @ ω%
d_{100} UNFROZEN
d_{100} FROZEN
t_{100} REM. @ ω_L
t_{100} REM @ ω%
t_{100} FROZEN
t_{100} UNFROZEN

PLATE 61.

General Conclusions

Tests performed for this thesis indicated:

1. The highly plastic preconsolidated clay showed a loss of strength upon thawing after being subjected to either rapid or slow freezing, but also exhibited a partial recovery of strength with time.

2. Sufficient data was not available in this program to definitely conclude that a loss of strength was obtained in the silty clay tested. However, similar results published by the Highway Research Board indicated a loss of strength in a silty clay.

3. No loss of strength was evident in the Bentonitic Shale tested, but visual inspection and tests indicated that freezing tended to segregate the shale and make it act similar to a granular material.

4. Consolidation characteristics of the clay were slightly altered by freezing action. Consolidation tests showed the frozen soil to act similar to identical soil which had been remolded at its natural moisture content.

Summary of Results

The following table gives a concise picture of the type of soil tested, conditions and the results obtained in the foregoing thesis, (Table I).

TABLE I

SUMMARY OF RESULTS

Test		Type of Soil	Type of Test	Freezing Conditions	Test Conditions	Test Results
RENGTH TESTS	SERIES I	Highly Plastic Clay	Unconfined Compression using 1 cm. size specimen.	Rapid	Tested immed. after thaw	Evidence of loss of strength.
				Slow	Tested immed. after thaw	Evidence of loss of strength.
	II	Silty Clay	Unconfined Compression 1.4 in.spec.	Rapid	Tested immed. after thaw	No evidence of loss of strength from limited data available
		Shale	Triaxial Compression	Rapid	Tested immed. after thaw	No evidence of loss of strength.
	III	Highly Plastic Clay	Unconfined Compression 1.4 in. specimens	Rapid	30 day recovery	Initial strength loss of 14°/o. Recovered 70°/o in 30 days.
					60 day recovery	Initial strength loss of 50°/o Recovered 10°/o in 60 days.
					90 day recovery	Initial strength loss of 50°/o. Recovered 20°/o in 90 days.
	IV	Highly Plastic Clay	Unconfined Compression 1.4 in. specimens	Slow	30 day recovery	Initial strength loss of 35°/o. Recovered 20°/o in 30 days.
					60 day recovery	No further recovery beyond 30 days.
					90 day recovery	No further recovery beyond 30 days.
	V	Highly Plastic Clay	Triaxial Compression 1.4 in.spec.	Rapid	Tested immed. after thaw	No loss of strength evident.
SOLIDATION TESTS		Highly Plastic Clay	One-dimensional consolidation	Rapid	Tested immed. after thaw	Compressive index is altered similar to remolding. Little change in permeability. Little change in rate of consolidation.

Recommendations for Future Research

1. Results from these studies indicate that there may be some correlation possible between loss of strength due to freezing and the Atterberg limit values of the soils tested. (Ref. Plate 51). Such a correlation would permit a rapid evaluation of the loss of strength characteristics that a particular soil might have upon freezing.

2. Plots of strength test results showed considerable scatter, the cause of which may be due in part to variations in soil types from different samples, and possibly due to variations in the degree of saturation of different specimens. It is felt that a soil sample for future work should be of the form of an undisturbed block, approximately 1 foot cubed, from which could be extracted many 1 cm. specimens of more nearly the same soil type.

3. In future work, the degree of saturation of the soil should be noted for each strength test, as this appears to have an important bearing on results. Test results could thus be compared at equal degrees of saturation, and it is believed by the authors that considerably less scatter of points on the log of strength versus moisture content plots would result.

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APPENDIX "A"

Test Procedure for Unconfined Compression Tests using the direct reading Fisher Balance type apparatus.

(Photo of Fisher Balance apparatus is shown on Plate 1).

Preparation of Specimens:

1. Small brass sampling tubes, 1 cm. in diameter and 1 $^{1}/2$" long, were used for cutting specimens from the samples.

2. The specimen was pushed to one end of the sampling tube by means of a plunger, the end of the specimen squared off with a spatula, and a small one cm. lucite cap placed on the end of the specimen. The specimen was next pushed to the other end of the tube, the above procedure repeated, and the specimen then removed from the tube.

3. The length of the specimen was carefully determined by means of a micrometer and the specimen, with a lucite cap on each end, then set up in the testing apparatus.

Test Procedure:

(The controlled stress type of testing was employed).

1. The jacking screw was turned down until about 5 ^{o}/o of the total load registered on the balance dial and this load maintained until the next load was applied.

2. Equal increments of load were applied every half minute on the minute and half minute and the Ames dial read every half minute on the quarter and three quarter minute.

3. Loading was continued until failure and a progressive plot of load versus compression was maintained in each case.

4. Immediately after failure, the specimen was placed in a small pre-tared moisture content container and the weight obtained to the nearest .005 grams on a micro balance. A sketch of the specimen and the type of failure was made for each specimen.

5. The specimen was next dried in the oven, cooled in a dessicator and weighed dry to the nearest .005 gram. A moisture content determination was thence made.

For specimens that did not fail at a load less than 500 grams the following procedure was used to change the balance dial.

1. Fasten the dial arrest on the dial yoke rod which is in front of the loading yoke so that the pin is resting snugly on top of the loading yoke. Secure by set screw.

2. Release the loading yoke, allowing it to slide up on the one quarter inch rods until the balance dial registers zero. Clamp in position by means of the set screws.

3. Release the dial arrest.

4. Move the left hand sliding weight to 500 grams.

5. Proceed to apply load by means of the jacking screws as before.

If the sample did not fail at 1000 grams the procedure as outlined under 4 was repeated, setting the sliding weight at 1000 grams etc.

APPENDIX "B"

Procedure for Unconfined Compression Test (1.4" diam. specimen).

The 1.4" specimen is obtained from material in the 3" diameter Shelby sampling tube in the following manner.

1. The Shelby sampling tube is mounted in a hollow framework in a testing machine. A 1.4" diameter specimen tube with cutting edge upper most is positioned centrally beneath the Shelby tube and just in contact with the soil sample. A plunger is now inserted in the upper end of the Shelby tube and the testing machine used to force the sample out of the Shelby tube and into and around the 1.4" specimen tube. The 1.4" specimen tube is filled to a length of $3^1/2$" or to give a length-diameter ratio of from 2 to 3. Surplus soil is removed from around the 1.4" specimen tube and preparation of the specimen is commenced.

2. The ends of the test specimen are levelled off by means of a straight edge and by pushing the specimen to either end of the 1.4" tube with a small plunger. (This operation may be by hand or by means of a jacking device or testing machine).

3. The specimen is pushed out of the 1.4" specimen tube and the length of specimen carefully measured. The specimen is then positioned for testing in the platform-scale type unconfined test apparatus.

4. Reduction in length of the specimen is measured by an Ames dial. The original reading on the Ames dial is recorded and the specimen

failed by applying load, in increments of 5^{o}/o of estimated failure load, to the scale pan. Corresponding Ames dial readings are taken for each increment of load. Loads are applied on the minute, and half minute and readings are taken on the 1/4 and 3/4 minute and readings are taken on the 1/4 and 3/4 minute mark.

NOTE: If a plastic type failure occurs, a stress strain curve should be plotted or the load at 20^{o}/o strain may be taken as the ultimate load.

APPENDIX "C"

Procedure for the Triaxial Compression Test (Quick Test, using 1.4" diameter specimen.

1. Preparation of the specimen is identical to that outlined in Appendix "B".

2. Slip a thin membrane over the pedestal of a standard triaxial test apparatus. Securely fasten this membrane to the pedestal with rubber bands.

3. Place the specimen on the pedestal and roll the membrane up around the specimen.

4. Disconnect the Saran tubing from the top loading cap and plug both top and bottom.

5. Position the top loading cap, roll the membrane up over the cap and secure with rubber bands.

6. Install the necessary rubber gaskets and place the lucite cylinder and top loading head in position.

7. Securely tighten the nuts on the rods through top and bottom loading heads and position the apparatus beneath the loading yoke of the triaxial machine.

8. The weight of the apparatus is balanced off on the triaxial machine by moving the sliding weight on the balance arm.

9. The Ames dial is set to read approximately .9000.

10. Lateral water pressure is next applied to the specimen and the balance arm once more zero'd.

NOTE: The same or almost identical zero readings on the Ames dial should be obtained before and after applying lateral pressure.

11. The specimen is now failed in an identical manner as described in Appendix "B".

12. The moisture content of the sample is measured at the end of the test.

APPENDIX "D"

Procedure for Consolidation Test

1. Record the consolidation ring number.
2. Determine and record weight of ring to .01 gm.
3. Determine and record average height and diameter of ring to .01".
4. Record machine data (weigh block + stone + ball to 1 gm).
5. Prepare the soil specimen as follows:
    (a) Extract specimen from Shelby sampling tube, approximately 2 ins. thick, directly into cutting ring.
    (b) With cutting ring as a mould, trim bottom of the specimen flush with mould using straight edge. The bottom will correspond to the sharp edge of cutting ring since Shelby tube is inverted during extraction.

6. Place glass plate under consolidation ring and gently push specimen from cutting ring into consolidation ring until the prepared surface of the specimen is in contact with the glass plate.

(a) Remove the cutting ring and, as before, trim the top surface of the specimen, buttering as necessary, so that the top surface is flush with the consolidation ring. Care must be exercised during preparation in order not to disturb the initial voids ratio.

(b) Place the other glass plate on the top of the specimen and weigh the whole, soil + ring + plates to 0.01 gm. Record the weight.

Perform the consolidation test as follows:

(a) Place on porous stone in the consolidation dish, then carefully place the consolidation ring and specimen in the centre of the stone. Centre the second porous stone on the specimen, then superimpose the block and steel ball on the top of the top stone.

(b) Place the dish in position such that the hole through the loading yoke is centred directly above the steel ball and not quite in contact.

(c) Place the Ames dial in position so that the stem is resting on the steel ball. It is necessary to know the length of travel of the dial stem to permit any re-set that may be necessary as a result of excess swelling or compression.

(d) Place a small weight on the scale pan. Quickly and carefully balance the beam of the platform scale. Observe and record the dial readings after elapsed times of 6, 12, 24 and 60 secs., 2, 4, 8, 15, 30, 60 mins. etc., continuously doubling the time interval between successive readings.

After the 2 min. reading, pour distilled water into the dish until the level of the water is above the bottom of the specimen (approximately half way up the ring). When swelling stops, apply water so that the top surface of the soil is immersed.

The magnitude of the initial load should be sufficiently small

so that any swelling may take place. An initial load of 10 gms. on the scale pan is usually sufficient. When the specimen is a highly swelling soil, greater initial loads should be used.

(e) Make progressive plot of dial readings vs. time on semi-logarithmic paper.

(f) After the primary swelling has taken place as indicated by the shape of the time curve, apply the next increment of load, balance the platform scale, and record the dial readings as indicated in (d) above. Each successive increment of load should be approximately equal to the previous total load on the pan. (To obtain a rebound curve the following pattern of loading was used, 10, 20, 50, 100, 200, 400, 800, 200, 50, 100, 200, 400, 800, 1500, 3000, 6000, 1500, 400, 100, 50, and 10 gms. on the pan, successively).

(g) Make a progressive plot of dial reading vs. pressure on semi-logarithmic paper. The load increments should be continued until the strain line portion of the curve is well established. (The maximum capacity of the platform scale must not be exceeded, - i.e. twice the rated capacity).

(h) Once a sufficient number of load increments have been applied, remove the load in decrements such that the load on the scale pan is one-quarter of the previous load. (Numerical values used in this case were given in (f) above). As before the dial readings should be observed and recorded at time inter-

vals given in (d) above. Time curves should be plotted for each decrement of load.

(i) After completion of the test, disassemble the apparatus, blot any excess water from the surface of the ring and specimen, place both in a dish, weigh and record the weight of tare + ring + specimen + glass plated to 0.01 gm.

(j) Dry the specimen in the oven at 110°C and record the dry weight of tare + ring + specimen + glass plates to 0.01 gm.

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